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MINUTES OF PROCEEDINGS
OF
THE INSTITUTION
OF
CIVIL ENGINEERS;
WITH OTHER
SELECTED AND ABSTRACTED PAPERS.

VOL. CI.

EDITED BY
JAMES FORREST, ASSOC. INST. C.E., SECRETARY.

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CORRIGENDA.

Vol. ci. p. 62, line 18, *for* "Tons of steel in grades," *read* "Tons of steel in girders."
„ p. 63, line 12, *for* "and the bore," *read* "and the base."
„ „ line 31, *for* "Ryland and Moore," *read* "Ryland and Morse."
„ „ line 32, *for* "Span C," *read* "Span No. 6."

THE INSTITUTION OF CIVIL ENGINEERS.

SESSION 1889-90.—PART III.

SECT. I.—MINUTES OF PROCEEDINGS.

4 March, 1890.

GEORGE BERKLEY, Vice President, in the Chair.

The following Associate Members have been transferred to the class of

Members.

Charles Edwards Willoughby Dodwell, B.A.
John Horan, M.E.
Henry Marsh.
Frederick Ewart Robertson.

The following Candidates have been admitted as

Students.

George Arthur Marshall Baker.
Thomas Barlow.
Victor Borrero.
Charles Hermon Craven Brady.
Herbert William Butler.
Patrick Joseph Corbett.
Harry Glen Finlaison.
Edwin George Gahagan.
George Tertius Glover.
John Heathcott.
George Forster Hickson.
Robert Holt, Wh.Sc.
William Hutton.
Charles Richard Judd.
Cooper Nichol Kirton.
William Eugene Marsily.
George Arthur Monkhouse.
George Emmanuel Nathan.
Bertram Edward de Noual Ramus, B.A.
John Lucas Rickard.
Andrew Williamson.
Francis Wood.
Henry Wootton.

The following Candidates were balloted for and duly elected as

Members.

Andrew Miller Alexander.
Robert Kaye Gray.
Alfred Bernard Phelan, B.A.

Associate Members.

Rayner Childe Barker, C.I.E.
Harry Oscar Beckh, M.A., Stud. Inst. C.E.
Alexander Bremner.
Charles Chambers.
George James Clarke.
James Alfred Cones.
Claude Francis Arthur Egerton, Stud. Inst. C.E.
Clarence Noël Goodall, Stud. Inst. C.E.
William Erskine Fraser Handcock.

Associate Members—continued.

Herbert Spong Hawkins.
James Allan Henderson.
Patrick Nicholas Hill Jones, Stud. Inst. C.E.
Frederick Jorgensen.
John Knight.
Tom Gallon Lumb.
William McKelvie.
Henry Allan Moss.
Joseph Moynan, B.C.E.
John James Myres.
Adolphe Ernest Orr.
Harold William Pauling, Stud. Inst. C.E.
Alfred Cecil Pereira, Stud. Inst. C.E.
Charles Roberts, F.C.H.
Arthur Samuel Francis Robinson.
Charles Tallent Spencer, Stud. Inst. C.E.
Robert Anthony Synge, B.E.
Alexander Thomson Walker.
Horace Reginald Waters, B.E.

(*Paper No. 2421.*)

"The Hawkesbury Bridge, New South Wales."

By Charles Ormsby Burge, M. Inst. C.E.

The railway system of New South Wales until recently consisted of two separate divisions, the one starting from Sydney, and branching in a westerly, southerly, and south-westerly direction, while the other originated at Newcastle on the sea-coast, about 100 miles north of Sydney, and communicated with the northern part of the Colony, and with Queensland.

The Hawkesbury Bridge is situated upon the connecting link of railway which was designed to unite these separate divisions. At the site of the bridge, about 7 miles from the sea, the estuary of the Hawkesbury has a total width of about 6,600 feet, and is divided into two channels by Long Island (Plate 1, Fig. 1). A solid embankment carries the railway across the comparatively shallow southern channel, on leaving which the line is tunnelled through a high promontory forming the eastern end of Long Island (Plate 1, Fig. 2), and then crosses the main channel to Mullet Point by the bridge which is the subject of this Paper.

The Colonial Government, after a considerable amount of discussion, decided to invite not only tenders for the construction of the work, but also designs for the same, and took the necessary steps to make the competition world-wide. A committee, consisting of Mr. W. H. Barlow, Past President Inst. C.E., Mr. G. Berkley, Vice President Inst. C.E., and Captain Douglas Galton, R.E., Assoc. Inst. C.E., was appointed to adjudicate on the tenders; and when the report of this committee had been supplemented by others from Sir John Fowler, Past President Inst. C.E., and Mr. John

Whitton, M. Inst. C.E., Engineer-in-Chief for Railways, New South Wales, it was finally decided to accept the tender of the Union Bridge Company, of New York, who undertook to complete the bridge within two-and-a-half years for the sum of £327,000.

The accepted design consisted of seven spans of 416 feet each, from centre to centre of the piers, the foundations for the latter being of concrete encased in steel caissons, while the upper portions of the piers and the whole of the abutments, are of masonry. The girders are formed of built steel compression-members, and solid steel eye-bar tension-rods, all the connections being made by steel pins. The cross-girders and rail-bearers are of riveted steel plate. The two main girders of each span are 410 feet 0½ inch long, from end pin to end pin, and 58 feet deep at centre, and are placed 28 feet apart, from centre to centre, the bridge carrying two lines of railway.

The general elevation of the bridge is shown in Plate 1, Fig. 3.

LOCATION.

The line and spans of the bridge were located by a system of triangulation carried out by Mr. F. Small, and continued by the Author who succeeded him as representing the Government, and by Mr. O. Schulze on the part of the contractors. The results obtained from the first base-line on the shore were afterwards supplemented and checked by using a portion of the structure itself, when fixed, as a secondary base-line.

FOUNDATIONS.

The borings showed a bed of mud extending to a depth varying from 60 to 170 feet below high-water mark, and overlying the sand; the greatest depth of water being 77 feet, and the range of tide 7 feet. The greatest depth of the foundations occurs at pier No. 6, which is carried down 162 feet below water, this being, as far as the Author is aware, the deepest bridge-foundation yet sunk.

The caisson for each pier is rectangular, with rounded ends, and its construction is shown in Plate 1, Figs. 4 and 4*a*. It is 48 feet long transversely to the bridge, and 20 feet wide, splaying out in the lowest 20 feet, so as to form a tapered shoe which is 2 feet wider all round at the bottom. In the centre line on plan, parallel with its length, are three wrought-iron dredging-tubes, 8 feet in diameter, and 14 feet apart from centre to centre. These are connected to the outer skin and to each other by steel strutting of tees and angles.

At the bottom, the dredging-wells splay out in a trumpet mouth so as to meet the outer skin, and also each other, in a strong cutting edge formed of heavy steel plates.

The method of sinking the caissons was as follows:—The shoe, having been built on shore at Dangar Island, and provided with a timber false bottom, was floated out to position and sunk to the bottom of the river, by removing the temporary bottom, and partially loading the caisson with concrete.

The caisson was then sunk through the mud by dredging the material from the bottom of the wells and by loading the space between the wells and the skin with concrete, more steel being built up as the caisson went down.

As soon as the structure was firmly in the sand, the dredging-wells were filled with concrete, and the masonry was then begun at a level somewhat below low-water.

The concrete was composed of 1 part of Portland cement, 3 parts of sand, and 6 parts of stone, broken to 2½-inch gauge.

The stone was what is locally known as Kiama blue stone, the material being mixed by Jamieson's concrete-mixers, each passing through about 5 cubic yards per hour.

The concrete in the shoe was made stronger by the addition of one-fifth of a cask of cement per cubic yard.

The caisson for No. 5 pier was the first one started, and, having undergone greater vicissitudes in its downward progress than any other, was the last to be completed. The sinking was begun on the 9th of December, 1886, and the foundation was only ready for the masonry on the 9th of October, 1888.

Shortly after it had well entered the mud, the caisson showed a tendency to work eastwards, that is to say, transversely to the direction of the bridge. Efforts to recover its position were first made by endeavouring to cant it eastward at the top, by excavating the eastern well in advance of the others, thus pointing the central vertical axis downward in the westward direction required; but this was not successful, as even when the eastern well excavation was 15 feet deeper than the western one, the cant was still westwards. Dredging outside was then resorted to without effect, also dumping the excavated mud outside on the eastern end. When the caisson had reached 75 feet below the river bed, the divergence had amounted to 5 feet at the bottom and 3 feet at the top, the cant still continuing westward, notwithstanding constant extra sinking of the eastern wells. The margin for lateral divergence allowed by the specification for the caissons was 2 feet.

A recovery of about 18 inches at the bottom was affected about this time, and when the west cutting edge first felt the indications of sand, the caisson, as had been expected, commenced a righting movement towards the vertical, but of course at the expense of still further lateral divergence at the top.

At this juncture, the 7-feet grab working in the eastern well, got caught in the bottom splay, and as a considerable period elapsed before it was recovered, work was suspended at this caisson for a time. Later on, two grabs were similarly caught in the centre well, one of which was never recovered—diving operations, at the depth then attained, being attended with great difficulty.

The contractors now began driving piles, at the east end, to sustain a cribwork, which was loaded with stone and was intended to form a buttress against the structure and prevent further movement; and, subsequently, a similar cribwork was put in about 400 feet upstream or westwards, with the intention of anchoring the top of the caisson firmly to it; and then, by digging persistently in the eastern well, it was hoped that the bottom would be brought back to its position. But as to all these proposals, it was always the Author's opinion, that, as the purchase, in each case, was in the same unstable medium of mud, any horizontal movement in the caisson could not be corrected by either tying it to, or resting it against, any other structure not founded in the sand. This opinion was verified, as, on further sinking, the top, in righting itself, pushed over the eastern cribwork.

Operations, as regards No. 5 caisson, were now suspended for some months, except for the recovery of the lost grabs, and the construction of the cribwork upstream, as mentioned above. This experiment having failed, a proposal was made to cease sinking, as the caisson was well in the sand at 144 feet below high-water of ordinary spring-tides, and then it was proposed to rectify matters by sinking an additional caisson at the west end. This addition was, in plan, something in the form of a crescent with rounded ends, the concave side of the crescent being meant to fit the rounded west end of the original caisson. It was made of steel plates. Two wells were provided in the new caisson, one at each extremity, the space between being loaded with concrete, and the bottom finishing in cutting edges similar to the original one. The ill luck of the pier attended this effort also, for when about 28 feet from the bottom, the wells caved in, under the pressure of the mud on the one side and of the original caisson on the other, so that further sinking became impossible.

As the additional caisson could not now be got either down or up, there was no alternative left but to commence the masonry at the west end, at as low a level as possible, viz., 12 feet 6 inches under the original masonry level, and to corbel out. With the aid of a cofferdam this corbelling was carefully carried out with solid stones of 7 to 8 feet in length, with a 9-inch overhang in each course, and, though adopted as a last resource, the centre of the column of masonry above, coinciding with the centre of the west girder, is well within the base of the original caisson; and the resultant line of the pressures of the pier and load passes very closely to the centre of the bottom foundation area.

The sub-contractor for the work attributed the eastern divergence to the fact that the mud, at the east end, was more consistent than at the west; hence the material displaced by the splay at the former end did not cave in immediately, but stood for a considerable depth without filling the cavity. The west displacement on the contrary being immediately refilled by the softer mud falling into it, greater resistance was felt at that end, and the caisson gradually tended towards the other end, where the chief resistance was only at its projecting cutting-edge.

This theory is supported by the fact already noted, that persistent dredging of the east well in advance was not followed by faster sinking of that end, which appears to show that the material at the west end was softer and therefore more easily pierced by the cutting-edge. There is no doubt that the outward splay of the caisson at the bottom was the cause of nearly all the difficulties connected with the sinking; as in caissons Nos. 1, 2, and 3, where an additional outer vertical steel wall was subsequently adopted, no difficulty in sinking correctly into place was experienced. This addition, which was in the form of a pocket, filled with concrete all round the caisson, carried up from the cutting-edge 24 feet high, is indicated by a broken line P on the drawing of the shoe in Plate 1, Figs. 5 and *5a*.

About three months after the launch of No. 5 caisson, No. 4 was towed out and sunk with much less difficulty to a depth of 147 feet below high-water of spring-tides, and with comparatively slight divergence from its correct position.

No. 6 was launched about six weeks after No. 4, and gave some trouble. Its site coincided with a sudden declivity in the bottom of the river. The caisson tended northward, following the declivity, and continued to do so, even when the sinking had progressed to a great depth, and in spite of the fact that large quantities of stone were dumped to the bottom, at the north side of the structure. The caisson also took a skew position, though not to such a degree as to interfere with the correct location of the masonry pier upon it, at right-angles to the bridge. When the caisson reached the bottom, at 162 feet below high-water of spring-tides, it had a slight lean to the southward at the top, notwithstanding that its proximity to the northern bank of the river had afforded facilities for counteracting this tendency by anchoring the top northwards.

Being unable to cope with the general northern movement of caisson No. 6, the contractors had to get the consent of the Government to increase the span of the girders, between the fifth and sixth piers, by 4 feet 3 inches. As the north abutment had not yet been commenced, its site was shifted 4 feet 3 inches north, so that the width of span No. 7 remains the same as originally intended.

The maximum pressure on the base of No. 6 caisson, which is the heaviest, is about 9 tons per square foot.

About three months after the launch of No. 6 caisson, No. 1 was moored in place, and, being constructed with vertical sides without any splay at the bottom, it was sunk without difficulty through the mud, which was here of no great depth. Before the caisson reached the bottom of the water, however, it met with an accident similar to that which had befallen the caissons of the Jubilee Bridge over the Hooghly,[1] and of the Poughkeepsie Bridge over the Hudson River. Each of the Hawkesbury caissons, in sinking, had been lashed to a 700-ton barque, which was used as a store, workmen's quarters, &c.; and on the night of the 18th of August, 1887, the wind and tide becoming too strong for the anchorages, the ship and caisson with the derrick punts attached, were all carried away some distance down the river, where, with some difficulty and risk of wreck, they were re-anchored. No attempt to tow the structure back was made till the weather settled, which was about a week subsequently. No. 1 was at last founded at about 101 feet below high-water of spring-tides.

No. 2 was launched about two months after No. 1, and, four months later, was founded accurately at 155 feet below high-water of spring-tides, being nearly as deep as No. 6.

Two months later, or at the end of November 1887, the remaining caisson, No. 3, was towed out, and founded, without incident, on the 16th of February, 1888, at a depth of 146 feet below high-water of spring-tides.

The foundation work was carried out by sub-contractors, Messrs. Anderson and Barr, of Jersey City, U.S.A.

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[1] Minutes of Proceedings Inst. C.E., vol. xcii. p. 79.

There can be no doubt, the Author thinks, that, having in view the recorded experience of the Hooghly, Poughkeepsie, and Hawkesbury bridges, this class of foundation is well suited for excessively deep work, and there is no reason why even the great depth attained at the Hawkesbury should not be exceeded if required; but this experience, especially at the latter work, points to certain essential features of design, the want of which gave much trouble.

In the first place, the bottom outward splay should be avoided. If it is desired to diminish skin-friction in sinking, the increase of size in the caisson shoe should be made by an offset, as in the amended form of caisson for piers Nos. 1, 2, and 3, shown in Plate 1, Figs. 6, *6a*, *6b*, *6c*, and *6d*, and not by tapering, as in the first three launched.

Secondly, as the wells, or dredging-tubes, are the only means by which the descent, in such large and deep caissons, can be regulated, they should be so distributed in plan as to facilitate this object. Fortunately, there was no serious deviation from the vertical in the ultimate position of the caissons described, but the tendency to cant longitudinally as regards the bridge, and transversely to the caisson, was found to be very difficult of control. The three wells, being in one line east and west, it was generally easy enough to depress the caisson to the eastward or westward, by dredging the east or west well; but there was no corresponding means of counteracting a northward or southward inclination of the caisson. When sinking had proceeded to a certain depth, the caisson could not be righted by unequal loading; while the digging in any of the wells tended only to increase the cant, the dredger swinging over to the vertical and excavating below, and undermining, the cutting-edge on that side which was already too low. The wells, therefore, should be at least four in number, and disposed in a diamond or quadrangular plan, so as to allow of each well acting separately for regulating the descent.

Finally, notwithstanding these precautions, the proportion between the size, in plan, of the top of the caisson, and that of the finished structure or pier to be raised upon it, should be sufficient to allow of a moderate amount of deviation from the true location in the former, without affecting the correct position of the latter.

The masonry, which was of a massive character, does not call for special remark. It was begun as each caisson was complete, and was out of hand in January 1889. It was carried out by a subcontractor, Mr. Louis Samuel, whose lamented death, during the currency of the contract, was widely deplored.

ERECTION OF THE SUPERSTRUCTURE.

Having in view the exposed situation of the bridge, and the strength of the tidal currents, the contractors decided to construct a floating pontoon, sufficient in size to carry a complete span, and to erect the girders upon it at an elevation high enough to command the piers.

The pontoon, when launched, was towed, with the staging upon it, to a sheltered bay at Dangar Island, and there scuttled on a gridiron in shallow water. In this position it remained resting on the gridiron while the span was erected upon it; then, at a favourable conjunction of wind and tide, the pontoon, with its load, was floated by closing the valves at low-water, and thus preventing the re-entry of the water when the tide rose. The whole was then hauled out to the bridge, about ¾ mile away, by 6-inch hawsers (Plate 2, Fig. 1), extending from the island to the bridge, running through steam winding-engines on the pontoon and on shore, while the propulsion was assisted by tug-steamers and by the flowing tide, Dangar Island being below the bridge. Being thus moored at high-water between the piers, the pontoon was allowed to fall with the tide (or in some cases assisted by partial scuttling), thus leaving the span in place. The pontoon was then taken back to the gridiron, for the erection of the next span. The chief risk, therefore, was confined to the navigation of the pontoon from the island to the bridge and back, an operation which required only a very few hours, and for which the time was carefully chosen for wind, tide, and weather.

As the abutments of the bridge were set back from the water's edge (Plate 2, Fig. 2), it was necessary that the pontoon for carrying the shore span should be made shorter than the girders, which overhung it at one end by a projection of 63 feet; and, as all arrangements for flotation and stability were provided for on this supposition, the remaining spans were dealt with in the same manner and were built on the staging with the same overhang. The overhanging end was in all cases intended to become the fixed end of the girder, and was first secured into place on the pedestals fixed on the bed-plates. The other end of the superstructure, which was about 5 inches higher, was then gradually let down upon the roller bearings and guided into position by vertical steel bars.

Details of the pontoon and staging, on which the superstructure was put together, complete, including floor-beams, wind-bracing, &c.,

excepting only the floor-beams of the projecting two panels, are given in Plate 2, Figs. 3 and 4. The two panels, or 63 feet of overhang, were temporarily strengthened, during launching, by timber struts and iron ties, to meet the contrary stresses to which, under the influence of the overhang, they were subject.

A fixed staging, not shown in the Plate, was erected at the overhang end of the pontoon, and separate from it, to support the cantilever end during erection, and a traveller, on rails 36 feet 6 inches apart, on the top of the staging, and commanding two complete panels, spanned the truss transversely. This traveller was run on to the fixed staging when a span was complete, leaving the latter free for launching. On the top of this fixed staging also, were placed the hoisting engines for lifting the members of the girders from the deck of the pontoon to their places.

The pontoon and staging were constructed mainly of Oregon pine timber, and the former was divided into forty-four watertight compartments connected by valves. The whole was laterally trussed on the Bollman system, by four 1⅛-inch steel cables secured to the four corners, and working freely about upright round posts at the opposite sides, as shown in Plate 2, Fig. 4. The whole of the submerged portion of the pontoon was covered with galvanized sheet-iron. Five large anchors were provided in case of emergency.

The gridiron, which was parallel to the shore line, was constructed with a transverse declivity riverwards of 3 inches in a width of 61 feet, to facilitate floating off, and with a grade of 11 inches in its length higher at the overhang end. This longitudinal inclination of the pontoon, which, of course, followed that of its bed, was reversed when afloat, the overhang depressing that end of the girder 5 inches below the other end. The girders were built on the staging with a camber of 12 inches, which was reduced to about 3½ inches when they were left on the piers.

The first span set up was No. 4, this being considered the easiest to navigate, and as the currents were moderate and all the conditions favourable, this span was easily placed in position in the course of a few hours on the morning of the 25th of May, 1888.

In floating out span No. 1, Plate 2, Figs. 5, *5a*, and *5b*, great difficulties had to be encountered owing to the action of violent currents and the proximity of the rocky beach. The span was launched on July 11th, and notwithstanding every effort, the pontoon grounded upon the rocks on a falling tide, and for some hours the vessel and her load were in danger of going to pieces.

Fortunately, however, the pontoon was floated off the rocks without any considerable damage, and the span was afterwards safely deposited in position.

Spans Nos. 3 and 2 were launched without incident on the 16th of August and the 8th of September, respectively, the latter having occupied only eighteen working days in erection on the pontoon.

Span No. 7, being that next the north abutment, was started from the gridiron early on the 5th of October, but an adverse wind rising, the main hawser parted when half way across, and it was considered advisable to anchor for the night, and telegraph to Sydney for more steamers. These arrived during the afternoon, and were lashed alongside so as to prevent risk of the anchors dragging during the night. The wind, however, fortunately moderated, and the operation was successfully completed at high tide the following morning.

Span No. 5 was put successfully into place on the 29th of January 1889.

Span No. 6 was started on the 1st of March. The weather being somewhat unsettled, and, owing to the parting of the hawser during a squall of wind, the pontoon swung round into a dangerous position, from which it appeared impossible to avoid a collision with the structure already fixed. But by a dexterous manœuvre, adopted on the spur of the moment, the span was saved from destruction by bringing it into place in the reverse position to that originally intended, and as the girders were in all respects quite symmetrical, the change in the plan of operations made no difference to the finished structure.

The superstructure was erected and launched by Messrs. Ryland and Morse, of Chicago, as sub-contractors to the Union Bridge Company.

It is a curious circumstance connected with this bridge that, though the successful tender was made by an American firm, the whole of the steel and iron, except only that of the eye-bar heads, was provided by the United Kingdom, where also it was manufactured.

The Author was in charge of the work for the Colonial Government from nearly its commencement, under Mr. John Whitton, Engineer-in-Chief for Railways, and the contractors were represented successively by Mr. M. Van Brocklin, and Messrs. Charles and Frederick Macdonald.

The bridge was opened for traffic on the 1st of May, 1889, by His Excellency Lord Carrington, Governor of the Colony, in presence of between seven hundred and eight hundred guests, including representatives from all the Australian Colonies.

The communication is accompanied by three sheets of tracings and a framed photograph, from which Plates 1 and 2 have been engraved.

(*Paper No. 2398.*)

"The Construction of the Dufferin Bridge over the Ganges at Benares."

By Frederick Thomas Granville Walton, C.I.E., M. Inst. C.E.

The bridge over the River Ganges at Benares was constructed by the Oudh and Rohilkhand Railway Company, to connect the line, already built from Benares to Lucknow, with the East Indian Railway. This is the fourth bridge over the Ganges which has been erected by the same Company, and by means of it a more direct route is supplied between Bengal and the provinces of Oudh and Rohilkhand. By it also a duplicate and more direct link is formed between Bengal and the Punjab.

The construction of the bridge was sanctioned by the Secretary of State for India in July 1879. The design was then discussed and arranged between Mr. Hederstedt, the Company's Chief Engineer in India, Mr. Batho, the Consulting Engineer in England, and Mr. W. H. Barlow, Past President of the Institution.

The weights of the platform and the moving load allowed for are as follows:—

Moving load	1·600	ton per lineal foot of bridge.
Ballast	0·750	" " "
Rails, sleepers, &c.	0·050	" " "
Platform structure	0·456	" " "
	2·856	
Main girders	1·690	" " "
Extension	0·300	" " "

The allowed strain on the steelwork was 6·5 tons per square inch in the bottom booms and diagonals, 6 tons in the top booms, and 5 tons in the verticals and other long unsupported compressive members.

The bridge was commenced on the 19th of January, 1881, was tested on the 24th of September, 1887, and was formally opened by His Excellency the Viceroy, the Earl of Dufferin, on the 16th of December, 1887, receiving the name of the "Dufferin Bridge," in commemoration of his Viceroyalty.

The site selected by Mr. Hederstedt for the bridge is immediately below the town of Benares, which has existed for many centuries on the left bank of the river. The town is built upon a thick bed of clay, against which the river impinges and is thus restrained into a channel which is probably narrower and deeper at this point than at any other part of its course. The clay projects below the river-bed as far as the middle of the present channel, and upon it the left abutment and the three adjoining piers of the Dufferin Bridge are founded. The remainder of the site is entirely of sand which extends for many feet below the river-bed.

At the site of the bridge the original level of the left bank appears to have been just above highest flood-level, but it has been artificially raised between 30 and 40 feet by the debris of the old city, which formerly extended considerably below this point. This debris consists chiefly of clay, bricks, and tiles used in the constant rebuilding of houses, and is almost as solid as the original bank.

The right bank is about 7 feet below highest flood-level, and is subject to inundation for a distance of 5 or 6 miles inland.

The Ganges is joined by the Jumna at Allahabad, about 100 miles above Benares; and the water-level below the junction is chiefly affected by the fluctuations of the Jumna, which being fed by rapidly collecting streams in the highlands of Central India, often brings down sudden and extensive floods during periods of heavy rainfall in the collecting area. The rise of water from lowest level to highest flood in the Jumna is 49·25 feet at Allahabad, and at Benares 50 feet; whereas at Cawnpore, situated on the Ganges, 120 miles above the junction of the Jumna, the total rise is only 19 feet.

Floods, however, only occur during the rainy season, from June to October; and with the exception of slight rises of a few feet in January and February, the stream at Benares may be depended upon to fall gradually from the cessation of the rains in October, until their recommencement in June. During the dry season the current gradually lessens until its velocity is not more than a mile an hour, and the channel then silts up considerably; but during the rains the velocity of the current is occasionally 15 miles an hour, and the bed is sometimes scoured to a depth of 70 feet below low-water level—which gives a possible depth in full flood of 120 feet. Hence, in undertaking works below high flood-level, it is necessary to fix upon a programme for each season, and to work during that season at such high pressure as will insure the programme being so far carried out that all work in the river-bed shall be secure from the effect of floods.

All piers must be sunk so deep as to be secure against scour. Temporary works of any kind, if left within the action of scour, would inevitably be lost. Much valuable time was therefore spent in removing staging and plant at the beginning of the rains, and in restoring them at the beginning of each working season. This disadvantage is, however, common to most Indian rivers.

The greatest depth of the river in the dry season, when the works were commenced, was 37 feet, and the depth during highest flood was 100 feet; since the piers have been completed these depths have increased to 65 and 120 feet.

The bridge consists of seven spans of 356 feet and nine of 114 feet from centre to centre, and is built for a single line of 5 feet 6 inches gauge. The railway is carried between the main girders of the large spans, and on the top of the girders of the smaller spans. The height from low-water to rail-level is 79 feet, and the clearance under the main spans at highest flood is 25 feet, Plate 3, Fig. 1.

A metalled road, 22 feet in width, is provided at rail-level, and is used for cart traffic between the passage of trains. A footway 5 feet wide on cantilevers, outside the girders, is provided on each side. Both abutments are built as block-houses to contain troops for the military defence of the bridge, Plate 3, Figs. 2, 3, 4, and 5.

The bridge was commenced simultaneously at the two abutments, the distance between these being 3,507 feet 3 inches. It was important, therefore, that the measurements across that distance should be made with accuracy. To do this two lines were ranged parallel with the centre line at 85 feet distance on each side of it, and on these lines all the measurements were made during the construction of the bridge, at which time the centre line was continually obstructed by staging, &c.

The measurement on land was made with steel tapes; that across the channel, a distance of 1,800 feet at low-water, was made first trigonometrically, and afterwards by a fine steel wire which was stretched across the channel over pulleys exactly 20 feet above water-level, and strained by weights until the wire just touched the water at the middle of the span. Fixed points on each bank were then plumbed up to the wire, and solder marks made to correspond with them. This wire was then carried ashore, and fixed in the same way over a truly levelled piece of ground. The distance thus measured differed from that calculated by only 1 inch.

A transit instrument was fixed behind each abutment on the centre line by which the true alignment was maintained.

When fixing the girders of the third main span, which was the last to be erected, there was found to be only $\frac{7}{16}$ inch excess measurement in the entire length of the structure.

River-Piers.—The piers carrying the 356 feet spans, are each founded on a single well of elliptical shape, 65 feet across the major axis, and 28 feet across the minor axis, Plate 3, Figs. 8 and 9. Five of these wells were sunk in water varying in depth from 7 to 20 feet. At piers Nos. 1 and 5, where there was only a depth of 7 and 12 feet of water respectively, the wells were started on earth banks thrown into the river above water-level.

For each of these large piers iron caissons were provided having three internal excavating-chambers divided by cross walls. Three of these caissons were 10 feet in height, two of 26 feet, one of 42 feet, and one of 50 feet, to suit the varying depth of water in which the piers had to be built. They were constructed with the inner shell inclining inwards from the base of the outer shell at an angle of 45° up to a height of 6 feet, thus forming a cutting-edge, which was strengthened by a cast-steel shoe $1\frac{1}{2}$ inch thick. The cross walls in the caissons were not carried down as far as their base, but commenced at a height of 6 feet, they also being provided with cutting-edges. The caissons were constructed with iron plates $\frac{1}{2}$ inch thick up to 6 feet in height, and beyond that $\frac{3}{8}$ inch thick, the inner and outer shells being connected by angle bracings both horizontal and diagonal (Plate 4, Figs. 1 and 2).

The wedge-shaped portion of the caisson above the cutting-edge was filled in with cement concrete, and above that height the space between the inner and outer shells is filled with brickwork in mortar. Above the top of the caissons an iron crating is erected, formed of angle bars, 3 inches by 3 inches by $\frac{3}{8}$ inch placed 6 feet apart, and attached to the middle of the bars connecting the inner and outer shells, the vertical angle bars being connected horizontally every 8 feet in height. By means of this crating the brickwork between the shells of the caisson is connected with the upper brickwork, which is carried up above the caisson with the same section. In some of the later piers, however, the thickness of the upper brickwork was increased by corbelling over, this change having been adopted as the result of experience.

The caissons of piers Nos. 1, 6, and 7, which were all built on the dry bank of the river or on artificial islands, were sunk by hand excavation as far as the water-level. The sinking was in general continued below water-level by dredgers, unless, as was the case in piers 1, 2, and 3, a clay stratum, met with at varying depths, necessitated further excavation by hand. The caissons of piers 2, 3, 4, and 5, were constructed on and lowered from the pontoons, and were sunk by means of Bruce and Batho's dredgers working on staging erected on the pontoons.

The pontoons were of iron, four in number. These were built on a bank thrown out into such a depth of water as would ensure the vessels being near the water when they were ready for launching. They were launched sideways on six timber ways. Each pontoon was 200 feet long by 25 feet wide, and 10 feet deep, and was built with five water-tight bulkheads, and fitted with sluice valves. It was originally intended that the pontoons should be utilized in carrying the main girders direct into position from a site where they had been previously erected ashore; but this plan was eventually abandoned.

The pontoons were coupled together in pairs by girders, leaving 32 feet interval between the vessels. At each end of the double pontoon there were three of these connecting girders, the first being 35 feet from the centre of the vessels, and the other two at 10 feet intervals beyond. The girders were covered with a deck, leaving an opening 68 feet by 32 feet, in which the caissons could be built and lowered into the water. On the pontoons was erected a timber staging, 57 feet in height, carrying three overhead travellers, for working the diggers used in excavating. Each digger was passed over the centre of the corresponding excavating chamber in the pier. The details of the staging are best seen by reference to Plate 4, Figs. 3, 4 and 5.

A pair of pontoons having been moored approximately in position for commencing one of the piers, the first 10 feet of caisson was built upon a false flooring of timber, and when caulked was raised from the floor by four chains passing under the cutting-edge and secured by shackles and eyes riveted to the inner side of the caisson. The floor was then removed, and the suspended caisson was gradually lowered as its construction proceeded, the weight being augmented as required by the concrete and brick filling, until the cutting-edge approached the bed of the river. The caisson was then carefully adjusted in position, and when grounded, the chains were released by divers, and the brickwork being then carried up to about 12 feet above water-level, the pier was ready for sinking.

The diggers were 8 feet in diameter, weighing 3¼ tons when empty, and containing 130 cubic feet of earth, equal to about 6 tons more. They were worked by steam-travellers, capable of lifting 20 tons—the excess of power being necessary for pulling the diggers out of the ground. The position of the pier while sinking had constantly to be checked, and its level adjusted by working the diggers most on the higher side, but as a rule, very little trouble was experienced in keeping the wells level, and in position.

Piers Nos. 6 and 7 were, however, an exception, for, in sinking through a hard stratum about 28 feet thick, the wells got a cant over towards the south. It was on this side that the steam-hoists were placed which worked the dredgers, and it was found that the continual shake and pull of the hoist chains, gave the well a tendency to lean towards the hoists. No. 6 was 4 feet 9 inches out of the perpendicular when 109 feet in the ground, and No. 7 was 5 feet 3 inches out when 135 feet in the ground.

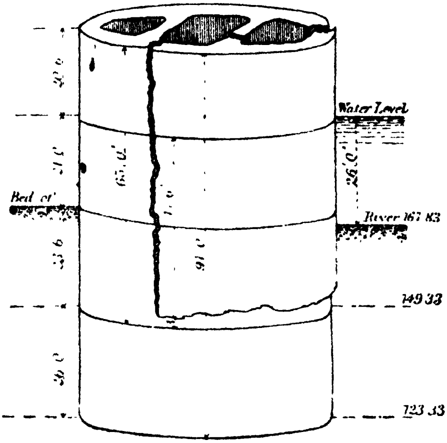
The steps taken to right these wells were as follow:—The steam-hoists were removed to the side to which the wells required to be drawn; a weight of about 1,000 tons, in rails, was placed on the top of the well, in such a manner as to bring the greatest weight to bear on the high side. The earth outside the well was then excavated, on the side towards which the well was leaning, down to water-level, leaving the slope of the excavation at 1 to 1. This slope was covered with wooden sleepers at intervals, and on these, in the direction of the slope, were laid rails 6 feet apart. Again, on these rails were laid sleepers close together, and the whole excavation was then filled in with bricks, forming a large wedge-shaped mass pressing against the side of the well continuously as it sank. In this way the wells were drawn over towards the vertical. No. 6, which was 4 feet 9 inches out of the vertical, was brought over that distance while sinking only 5 feet 8 inches, and No. 7, which was 5 feet 3 inches out of the vertical, was corrected in sinking 20 feet 6 inches further.

In sinking with the large diggers, the usual progress was 2 feet in twenty-four hours, the diggers being worked at night by the aid of electric light, and the masonry proceeding in the day; but the rate of progress varied according to the nature of the ground.

In the case of pier No. 4, the average rate of sinking amounted to 0·86 foot per day, and the average depth sunk per working day of the diggers was 1·43 foot; but the work was seriously delayed by an accident that occurred on the 17th of April, 1883.

The well had been founded on a caisson 26 feet in height, and at the time had been built up to a height of 91 feet, the lower edge being 70 feet 6 inches below water, and 50 feet below the bed of the river. The brickwork had all been built within two months, and was laid in ordinary mortar. The cutting edge of the caisson was resting on a clay stratum into which the diggers had excavated holes about 9 feet below it, and as it was feared that the clay might at any time give way and cause a sudden sinking of the pier, it was determined to build up the brickwork higher than usual above the water-level. The dredging was then recommenced, but immediately afterwards the new brickwork was burst outward by a sudden rise of the water inside the well, produced by the falling in of the clay.

Fig. 1.



SKETCH SHOWING FRACTURE OF NO. 4 PIER.

The fracture (*Fig. 1*) was found to extend downwards to within 3 feet of the top of the caisson, or 21 feet below the river-bed; and a mass of brickwork, 62 feet in height by 46 feet in width, was detached from the well and leaned outwards with a slope of 1 in 15, while the angle-iron crating built up in the brickwork had been torn through from top to bottom.

At that season of the year, when the rains were due within two months, nothing could be done to repair this damage, but it was proposed to get rid of the injured portion of the brickwork by explosives, and afterwards to fill in the gap by means of a double iron shield lowered from the top so as to lap over the sound brickwork, and afterwards filled with cement concrete.

However, after the rains it was discovered that the detached brickwork had fallen away, and the proposed blasting was thus rendered unnecessary. The shield was therefore constructed at the bridge workshops, and was lowered by winches from a staging, erected over the gap, the sinking of the shield through the bed of the river down to the base of the fracture being done by dredgers working inside it. After properly cleaning the surfaces of the brickwork, the shield was filled in with cement concrete, and the top of the patch was strongly bonded to the old work by rails. The well was afterwards sunk 70 feet 6 inches deeper, and is now the deepest foundation in the world.

The cost of the repairs was Rs.13,000.

The sinking of piers Nos. 1, 2, and 3, had to be carried out in a different manner, owing to the strong bed of clay met with. No. 1 pier was sunk only by hand-excavation, being entirely in clay, and therefore easily kept free of water. It was found, however, that the caisson was unable to withstand a greater pressure than that due to an external head of 42 feet of water, and in sinking Nos. 2 and 3 the hand-excavation was therefore only continued for a portion of the depth, the sinking being completed by divers.

When each well had been sunk to its final depth, its excavating-chambers were filled in with concrete (lowered in self-depositing skips) up to 5 feet from the top, the remainder being filled in with brickwork. It was found, however, in the case of the first pier, that the weight of the pier-superstructure caused a settlement of nearly 3 inches in the well, and the plan was then adopted of filling in the lower part of the caisson, up to the top of its conical portion, with clean sand before depositing concrete. The sand settled more closely under the slopes at the base of the caisson than could be the case with concrete, and much reduced the settlement in the other piers.

The weight of a main pier with its full load is, approximately, 16,000 tons, and the area of its base is 1,430 square feet. This gives a pressure of 11·19 tons per square foot; but the effective pressure may be considered as reduced to one-half of this amount by water displacement and side friction.

The nine smaller piers for the land-spans were each founded on two wells 12 feet 6 inches in diameter on iron curbs placed 25 feet apart from centre to centre. The steining of these wells was 3 feet thick. The sinking was done with Bull's dredgers, worked by hand, the piers varying in depth from 67 to 157 feet below ground-level (Plate 3, Figs. 1, 6, and 7).

At 5 feet above low-water level, these wells were connected by a corbelled arch, and the pier was built on the top. The piers have parallel sides and semi-circular ends, being 35 feet long, and 10 feet in width.

The main piers are 72 feet 6 inches in height above low-water. The well-foundation is built up to 5 feet above low-water, and finished off with a rough-faced stone plinth; on this the pier is built, elliptical in plan similar to its foundation, but having its major and minor axes 3 feet shorter, thus giving 1 foot 6 inches to spare all round the pier to allow for inaccuracy in sinking. No pier is more than 6 inches out of its true position.

The quantity of brickwork in the entire structure is 1,876,289 cubic feet, equal to 69,492 cubic yards; and the cost of that built in Kunkur-lime mortar was Rs.23 10a. per 100 cubic feet, and that built in cement Rs.46 10a. per 100 cubic feet.

The cost of the main piers, as they stand, was Rs.757,988, which gives an average of Rs.108,284 for seven piers sunk an average depth of 102·17 feet below low-water.

The cost of the extension piers was Rs.120,241, which gives an average of Rs.13,360 for nine piers sunk an average of 69·18 feet below low-water.

Work in the river-piers was much facilitated by the use of a floating timber gangway, 4 feet wide, across the river. Along this the whole of the material required in the river-piers for the masonry and concrete, amounting in all to 80,000 tons, was carried on coolies' heads, and the highest rate paid to coolies for carrying the material a distance of 1,200 feet, with a lift of 70 feet, was 4½ annas (or 6¾*d.*) per ton.

Girders.—The girders are entirely of steel. Those of the larger spans are of compound triangular type, 355 feet 6 inches in length by 35 feet 4¼ inches in depth, with uprights spaced 11 feet 8 inches apart, from centre to centre, and connected by diagonals, each crossing three bays at an angle of 45°. Cross-girders are riveted to the bottom beams at each upright, with cantilevers for the footway on the outside.

The girders of the smaller spans are 113 feet 8 inches in length by 11 feet 5¼ inches in depth. The uprights are spaced 11 feet 1½ inch apart, and single diagonals connect the top and bottom of each bay. The whole platform is covered by steel buckle-plates.

The weight of a pair of main girders is 491 tons, and that of a main span complete is 746 tons. The dead-weight placed on a main span, including permanent way and metalling in the road and footway, is 224 tons.

The weight of a pair of the smaller girders is 42 tons, and that of a span complete is 127 tons—the total dead-load being 79 tons.

The main girders were erected by utilizing the girders of the extension spans as a platform. These girders being 113 feet 8 inches in length, the main span could be temporarily bridged by three pairs of them supported by two temporary piers (Plate 3, Figs. 10 and 11).

The temporary piers were formed by a cluster of twelve 6-inch solid iron screw-piles, pitched 10 feet apart, thus forming a rectangular pier 20 feet in width by 30 feet in length. The iron-piles were connected, at every 40 feet in height, by horizontal and diagonal bracing. A strong platform of steel-plate girders formed the top of each pier, and carried the 113 feet 8 inches girders. The pile screws were of cast-iron, 3 feet 6 inches in diameter, and were fixed from a timber platform thrown across the end of a pair of pontoons, on the deck of which were placed the winches for working the capstans.

In some cases the piers had to be placed in 65 feet of water, where they would manifestly have no stability unless artificially supported. The diagonal and horizontal bracing could not without serious difficulty be fixed below water-level, but it was practicable to fit a set of bracing on to the piles above water, and then to slide it down the piers as far as the first joint in the piles admitted. In this way the bracing was in some cases got down to 15 feet below water, and it was then tightened up by divers. The lower part of the pier was surrounded by a mound of sand, which was raised in the following manner:—

The river-bed was first flushed up to a uniform level by forming a bank of block kunkur (rubble lime-stone) around each pier, and at such a distance from it as to allow for the slopes of the mound. The space thus enclosed was then filled in with sand, which was brought off in punts and deposited where required by means of a 12-inch sheet-iron pipe, leading from the punt to such a depth as was necessary. In this way scour-holes 15 feet in depth were filled up, and a level platform was formed for the reception of the artificial mounds.

These were then commenced by throwing in sand-bags to form the base of the mound-slopes. As soon as a wall of bags had been raised 4 or 5 feet in height all round the pier, the enclosed space was again flushed up with loose sand delivered through the pipe; and the process was repeated until the mound was completed to the point where the bracing began. In some cases the mound thus raised was 45 feet in height.

The temporary piers having thus been got into position, the 114-foot span girders, which had been erected in the meantime on the top of the pontoon staging, were floated into position. The girders were then strengthened by timber diagonals and iron trussing, and a timber platform for carrying the travellers was erected. The temporary structure was further stiffened by steel-wire rope guys, secured to anchors up and down stream.

Of the main girders two spans were carted across the river over the bridge of boats, and were erected from the north bank. The other five spans were lifted, piece by piece, at No. 7 pier, and pushed forward as required.

The riveting was almost entirely done by hydraulic riveters supplied by Messrs. R. H. Tweddell and Co., a set of pumps and an accumulator being fixed on each bank. Many of the natives became expert in using the machines, though quite untrained previously. As the work progressed the men improved in smartness, and each span was more expeditiously erected than the last.

In No. 3 span, the time occupied in erecting the temporary staging, from the pitching of the first screw-pile to the commencement of the main girders, was seventy-seven days. The time occupied in erecting and riveting the main, and bolting on the cross girders, was twenty-seven days. The removal of the temporary staging, including the screw-piles, occupied twenty-eight days. The erection of one main span, and the removal of all the plant from the influence of the river, was thus done in one hundred and thirty-two days. It must be borne in mind that work was carried on by day and by night, and sometimes it was necessary to work incessantly throughout the twenty-four hours, by having three gangs, each working continuously for eight hours. Spans 1, 5, 6 and 7 were erected in one working season, and spans 2, 3, and 4 in the next.

The extension-spans were all erected and riveted on the ground alongside their respective piers, and were lifted into place by derricks. The erection of these spans complete, with their upper platform, but not including flooring-plates, occupied an average of five and a half days per span.

The main girders were built with a camber of 9 inches. This was reduced to 6 inches on the supports being removed, and to 4½ inches under the full dead-load of cross and intermediate girders, flooring-plates, permanent way, and metalling for the cart road.

The cost of erecting one of the large spans was Rs.57 per ton, and one of the smaller Rs.24 7a. per ton, including the cost of erecting and riveting-plant. The total cost of the steel superstructure complete, including cost of plant, permanent way, road-metalling, and erecting, was Rs.2,518,695, equal to Rs.371 per ton of metal in girders, amounting in all to 6,365 tons, or Rs.715 per lineal foot. This allows no credit for money realized by sale of plant.

The above rates are calculated on the estimated weight of the girders, but the invoiced weight as actually erected was 287 tons in excess, which would reduce the cost per ton considerably.

The total cost of the bridge was Rs.5,665,618, or Rs.1,608 per lineal foot.

In addition to the cost of the bridge itself must be considered that of the railway and road approaches, amounting to Rs.737,742; thus making the total cost of the extension across the Ganges to be Rs.6,403,360.

There were nineteen fatal accidents during the progress of the work; but a far larger number of hands was employed than would be the case on such works in Europe—the number at one time having been over seven thousand daily.

The bridge was tested with a train of two engines (each weighing 58 tons), and loaded ballast-trucks (each weighing 15 tons), covering one entire span, the engines being placed in the middle of the train, the total weight of which was 296 tons. The greatest deflection of the large spans under the train, running 20 miles an hour, was 1·45 inch, and the greatest oscillation was 0·30 inch. The small spans generally gave deflections of 0·70 inch, and oscillation of 0·05 inch.

Electric light was used regularly during the working seasons, the installation having been supplied by the Gülcher Company.

The Paper is illustrated by seven tracings, from which Plates 3 and 4 and the *Fig.* in the text have been prepared.

(*Paper No. 2401.*)

"The New Bridge of the London, Chatham and Dover Railway Company over the Thames at Blackfriars."

By GEORGE EDWARD WILSON CRUTTWELL, M. Inst. C.E.

GENERAL DESCRIPTION.

THE new St. Paul's Station in Queen Victoria Street was designed to provide additional accommodation, long needed, on the City side of the river, for the increasing traffic on the London, Chatham and Dover Railway.

Within the station are three terminal lines of way, and on the west side, adjoining the previously existing lines of the same railway, are two through lines, which form a junction with the old lines a little to the south of Ludgate Hill Station. The new lines are carried across the river by the new Blackfriars Bridge, Plate 5, Figs. 1 and 2, which forms the subject of this Paper.

The bridge is designed to carry, in all, seven lines of railway, and has a clear width of 81 feet between the parapets; but this width is increased to 123 feet at the northernmost span, where it was necessary to provide space for the platforms as well as for the lines of way.

In consequence of the great width, and the numerous cross-over roads upon the bridge, it was impossible to employ a construction like that of the older bridge, consisting of main girders rising above the rail-level, and therefore a system consisting of arched ribs beneath the rails was adopted.

The bridge has five spans; the shore span on the Surrey side being 183 feet; the centre span, and the Middlesex shore span 185 feet each, Plate 5, Fig. 4; and the second and fourth spans 175 feet each; the lengths of the three middle spans correspond with those of the old bridge alongside.

THE ABUTMENTS.

The cofferdams enclosing the abutments were each composed of a single row of sheet-piles of sawn pitch-pine, driven to an average depth of 25 feet below the foreshore, or about 7 feet below the foundations of the abutments. The cofferdams were set forward some 27 feet in advance of the foundations, in order to utilize them for the construction of wharves.

On the outside of each cofferdam a trench was dug 2½ feet wide by 3 feet deep, and filled with clay puddle. The piles being of sawn timber, caulking was unnecessary, except in a few places where leaks appeared on the occasion of the closing of the dam. These leaks were speedily stopped, and the excavations for the abutments were kept dry by means of a 10-inch pump worked by a 6 HP. engine.

The excavation for the abutment on the Middlesex side of the river was carried down, throughout its whole area, to a depth of 15 feet below Trinity high-water; and below this the excavation was carried 10 feet deeper in bays about 12 feet in width by 37½ feet long, the latter dimension corresponding with the width of the foundations. Each bay was filled with concrete before the excavation of the adjoining bay was commenced. The bed of the foundations, at a level of 25 feet below Trinity high-water, is about 1 foot beneath the surface of the ballast overlying the London clay. The surface of the clay is about 15 feet beneath the foundations of the abutment.

The abutments consist principally of Portland cement concrete, in the proportions of 8 parts of concrete to 1 part of cement for the lower portion, and of 6 to 1 for the upper; but the springers of Bramley Fall stone, against which the arched ribs abut, are backed up with brickwork in cement, the courses of the brickwork being set at an inclination square to the line of thrust of the arched ribs.

THE PIERS (Plate 5, Figs. 3 and 5).

The foundations of Piers 1, 2, and 3 (counting from the Surrey side), were each sunk within three rectangular caissons, spaced about 6 feet apart; whilst, for the long pier on the Middlesex side four caissons were required. Immediately above low-water level arches were turned between the separate portions of the piers built within the adjoining caissons; above the archways the piers are continuous.

The pier-foundations are all carried down to a depth of 46 feet below Trinity high-water, where they rest in the London clay; the depth below the river-bed varying from 16½ to 23½ feet.

THE CAISSONS (Plate 5, Figs. 7, 8, and 9).

The caissons are rectangular, varying in dimensions from 30 feet by 32 feet to 30 feet by 26 feet. They consist of a single skin of wrought-iron plate ½-inch thick, for the botto 7 feet, and diminishing to $\frac{5}{16}$-inch at the level of high-water. The lower portion, or permanent caisson, is, in each case, 21 feet in height; and above this a temporary caisson was carried up to a further height of 28 feet, this portion being bolted to the permanent caisson, and removed after the building of the pier inside was sufficiently advanced.

The caissons were stiffened vertically at the corners by angle-iron, and on the sides by tee-irons spaced at intervals of about 7 feet, and were further strengthened by temporary horizontal stiffeners placed about 4 feet apart. For the latter purpose wrought-iron girders and diagonal struts were at first employed; but, owing to the difficulty of removing, them under the great pressure, it was afterwards found better to employ timbering.

The cutting-edge of the caisson was of rolled-iron, stiffened by means of triangular gussets butting against the underside of the lowermost horizontal girder, and other gussets were placed above the girder to prevent its yielding upwards. The pressure on the lower gussets proving somewhat greater than on those above, it was found in some instances that the girder bent slightly upwards and allowed the cutting-edge to be forced inwards to some extent. This necessarily impeded the descent of the caisson, and after the experience gained in sinking the first caisson, the gussets of all the remaining caissons were strengthened. The bottom girder and gussets were designed to be left in permanently.

The upper horizontal frames were stiffened vertically by temporary timber puncheons fitted between them.

At the top of the permanent caisson thickening strips were riveted to the skin plates to give them additional width, and on these rested the temporary caisson.

The bottom of the temporary caisson consisted of inverted channel-irons, containing flat strips of vulcanized india-rubber, and the joint was tightened up by means of bolts spaced about 2 feet apart round the edges of the caisson.

The vertical and horizontal stiffeners of the temporary caissons were generally similar to those of the permanent caissons although of a lighter description.

Erecting and Lowering the Caissons.

For erecting and sinking the caissons a staging was constructed around the site of each pier.

After the permanent caissons had been riveted together and the temporary girders fitted inside them, two pairs of trussed beams were placed above each caisson, with their ends supported on the staging at the sides. Between each pair of beams, near the four corners of the caisson, was placed a suspension or lowering rod, 2¼ inches in diameter, which passed down inside the caisson, and was secured by a cotter under the bottom horizontal girder. The latter was stiffened by extra gussets at this point. The rods were in lengths, coupled together by pins. The top length of each rod was 2½ inches in diameter, and on it a square thread was cut for a length of 6 feet from the top.

When the rods had been fitted in position, the caisson was lifted clear of the low-level platform on which it was built by means of the screwed rods, and the platform being removed from beneath, the nuts were turned in the opposite direction, and the caisson was lowered steadily down. To ensure uniformity in the descent of the four corners, spanners were provided for each of the four screws, and ropes attached to the spanners were passed round snatch-blocks, and connected to a movable crane which travelled along the stage at the side of the caisson, and thus turned each of the spanners by the same amount. The crane then returned to its starting point, the spanners and ropes were re-adjusted, and the process was repeated until the lowering had brought the ends of the screwed rods nearly down to the nuts.

It was then necessary to couple on another length to the lowering rods. This was performed by taking the weight of the suspended caisson upon four large wrought-iron hooks passing under the cutting-edge and connected to rods passing up outside the caisson, between the trussed beams. These rods were coupled in lengths and furnished with screws at their upper ends similarly to the rods inside the caisson.

As soon as the caisson reached the bed of the river (which had been previously dredged so as so give a level bearing for the cutting-edge), the outer rods were detached, and the hooks were removed from beneath the cutting edge by divers.

The inner rods were lengthened as the sinking proceeded, and were retained until the caisson was lowered to its full depth; as they were found indispensable for controlling the caisson during its descent. Apart from the duty of keeping the caisson level, the rods, by allowing one side or one corner to take the lead, afforded the means of regulating the lateral position of the caisson with great nicety.

The temporary caisson was added during the process of sinking, as soon as sufficient depth was gained to admit of the sections being built beneath the suspension beams.

EXCAVATING INSIDE THE CAISSONS.

The silt, ballast, and the upper portions of the clay inside the caissons, were excavated by means of Priestman's grab machinery, the grabs being worked by movable jib-cranes. The quantity excavated at each "grab" varied from ¼ to ¾ cubic yard, and when in full work about fifteen grabs were made per hour.

On account of the girders and timbering inside the caissons, the grabs could not approach their sides and corners, and it was necessary to employ divers to shovel the material towards the middle.

The caissons were weighted with kentledge to facilitate the sinking, and under this load the cutting-edge penetrated at first below the excavation; but as the sinking progressed and the friction increased, it became necessary to excavate down to the cutting-edge; and in some cases it was with difficulty that the caissons were forced down to their full depth. This was especially so in the case of the caissons of pier No. 4, which had to be sunk through thick layers of septaria embedded in the London clay.

To aid the descent of the caissons of this pier, hydraulic presses were applied, the purchase being obtained against the undersides of inverted trussed beams laid across the caissons, and secured to the piles of the staging surrounding the pier. When the presses at each corner of one of these caissons were exerting a pressure of 30 tons each, or 120 tons together, the total load on the caisson amounted to 390 tons, the hydraulic pressure being aided by the weight of the caisson, including timbering, &c., which was estimated at 143 tons, and also by the weight of the kentledge, amounting to 127 tons.

The least pressure exerted was in the case of one of the caissons of pier No. 2, where the caisson, including timbering, &c., weighed 108 tons, and the kentledge 117 tons, or a total of 225 tons.

As soon as the caissons had been sunk well into the clay they were pumped free of water, and the digging was continued. At first it was found necessary to carry on the excavation only at low-water, and to admit water into the caisson as the tide rose; but as the caissons penetrated to a safer depth into the clay, the time of working was extended, until, on reaching the finished bottom of the foundations, the tide was kept out altogether, and the concrete work inside was then commenced.

In the case, however, of three of the caissons of pier No. 4, and the easternmost caisson of pier No. 3, it was unsafe to allow the full pressure of water at high-tide on account of the leakage which then took place through the clay; and the sluices had therefore to be opened at each tide until the concrete had been brought up 3 feet above the bottom, and the leaks had been overcome.

The average depth through which the caissons were lowered, whilst the grabbing and divers' work was in progress, amounted to about 4 inches daily; but when the pumping-out and excavating was commenced, the average daily descent was increased to about 15 inches—the maximum in any single day having been 2 feet 9 inches.

The descent of the caissons was greatly impeded by layers of septaria which were met with at intervals in the clay. The average thickness of these layers was about 6 inches, and the separate boulders of which the layers consisted measured from 2 to 4 feet across.

At pier No. 4, where the depth of the silt and ballast overlying the clay amounted to 14 or 15 feet, a water-jet was used for displacing the material round the edges of the caissons—thus obviating much of the divers' labour in digging and shovelling. The jet was driven by a steam force-pump, through a 2-inch flexible tube, and through a nozzle $\frac{3}{4}$-inch in diameter, which was directed by the diver. The daily rate of progress by this method averaged about 6 inches, as against 4 inches in the case of the ordinary digging by divers.

The caissons were in every case cleared of water by Woodford's centrifugal pumps, and their working proved satisfactory.

CONSTRUCTION OF THE PIERS.

The foundations of the piers inside the caissons consisted, for a height of 17 feet above the bottom, of solid cement concrete in the proportion of 6 parts of concrete to 1 part of cement, in layers corresponding in thickness to the height between the frames of the caissons.

The concrete having been raised to within about 3 feet of the underside of any one of the temporary horizontal frames, blocks of stone were bedded in it about 4 feet away from the sides of the caisson, and at intervals of about 6 feet apart; and between these and the angle-iron shelf which supported the frame, raking struts of timber were jammed tight by means of folding wedges. These struts prevented the sides of the caisson from being forced inwards on the withdrawal of the temporary frame, which was effected as soon as the concrete had been brought up to the underside of the frame. The raking struts were left in permanently; being of small scantling (about 5 inches by 5 inches) it was considered that they did not impair the efficiency of the concrete foundations to any appreciable extent.

It was at first attempted to withdraw the frames without inserting these struts, merely trusting to the concrete to resist the pressure from the outside; but it was found, even after the concrete had been left to set for four days, that its upper surface was dislodged round the edges, thus allowing the sides of the caisson to be forced inwards to some extent. A great saving of time was therefore effected by inserting the struts, as the frame could be drawn at once, instead of waiting a week or more for the concrete to become sufficiently hard to resist the pressure.

In this manner the concrete was brought up frame by frame, until, at a level of 4 feet below the top of the permanent caisson, the brickwork was commenced. This was butted tight against the skin of the caisson up to the underside of the top frame of the permanent caisson, and on the withdrawal of the frame its place was made good with brickwork.

At the level of the top of the permanent caisson, or 25 feet below Trinity high-water, a set-off of the brickwork occurs, and at this level the granite facing begins. The method of building was as follows:—A course of masonry having been laid beneath a frame, and the backing put in, and the whole having become sufficiently set, the skin of the temporary caisson was strutted against the masonry, thus enabling the diagonal struts of the frame next above to be removed ready for the carrying up of the next course of masonry past the frame.

In this manner five or six courses of masonry, extending to a height of 6 or 8 feet above low-water, were set inside the temporary caissons; but at the ends of the adjoining caissons, where the connecting arches had to be turned, only three courses of masonry, rising just above low-water, were set.

The work having been brought up thus far, the temporary caissons were removed, and the connecting arches, together with the remainder of the work below high-water, were put in by tide-work.

On account of the bed-plates of the ribs being set back inside the face lines of the pier, none of the work between or above the bed-plates could be built until after the ribs had been finally fixed in position. Before entering upon the manner in which this operation was carried out, a brief description of the ribs themselves and of the other parts of the superstructure will be given.

THE IRON SUPERSTRUCTURE (Plate 5, Figs. 4 and 6).

The arched ribs are of segmental form. The width of span varies as stated at the beginning of this Paper, but the rise of all the spans is the same, viz., 18 feet 6 inches, which is equivalent to from one-ninth to one-tenth of the spans. The springing-level is 7 feet 6 inches above Trinity high-water, and the headway at the centre of each span is 26 feet at high-tide.

The ribs, Plate 5, Fig. 10, are of a uniform depth of 4 feet 6 inches, and the web-plate is everywhere 4 feet 2 inches deep. In order to obtain this similarity of depth with a varying strength of rib in due proportion to the strains, the areas of the flanges at the different sections in their length are modified, not by altering the number of the flange-plates, but by altering the widths of some of them. At the quarter spans, where the greatest strain is caused by unequal loading of the bridge, the flanges are each made up of four $\frac{3}{8}$-inch plates 18 inches wide; but at the ends and centres of the spans, where less strength is requisite, there are only two plates of the above width, and the remaining two are 9 inches wide, the area of the flange-plates in the latter case being thus only three-fourths of the area in the former case.

At the sides of each flange iron angle-bars are riveted, which add both to the appearance and to the strength of the ribs.

At the back of each heel-plate is a wrought-iron forging turned at the back so as to bear upon the half-circumference of a cylindrical pin of wrought-iron 8 inches in diameter, which rests in a semi-cylindrical socket, forming the front portion of the bed-plate. The heel-forging, cylindrical pin, and socket of the bed-plate are each 1 foot 7 inches in width, so that the bearing area upon the pin normal to the line of thrust is 152 square inches. As the maximum thrust that can occur along a single rib is 270 tons, the maximum pressure per square inch of bearing is thus about $1\frac{3}{4}$ ton.

The thrust is distributed by stiffened wrought-iron plates upon a rectangular back-plate 4 feet in height by 4 feet 6 inches in width. In order to give the plate a uniform bearing over the springers, it was packed off about $\frac{1}{8}$ inch clear of the latter, and fine cement grout was poured in behind. With the same thrust as given above, the maximum pressure at the back of the bed-plate amounts to 15 tons per square foot.

The ribs being free to pivot about their cylindrical bearings at either end, the forces acting along the flanges and web could be accurately defined, and a more economical distribution of metal ensured than in the case of a rib taking its bearings over the greater portions of its end surfaces.

The web-plate is of a uniform thickness of $\frac{1}{2}$ inch; but at the ends, where the flanges converge towards the centres of the heel-plates, an additional web-plate is riveted on at each side, and over these a tie-plate connects the points of divergence of the flange-plates.

The arched ribs are sufficiently stiff in themselves for the loads they have to bear, and there are no braced spandrels between them and the upper longitudinal girders carrying the flooring of the bridge.

The transverse intercostal bracings between the ribs occur at about every 10 feet throughout the length of the ribs. The top member of the bracing consists of a pair of continuous angle-bars, riveted to the top flanges of the ribs; and the bottom member is also a pair of angle-bars rendered continuous by being riveted to bars passing through holes cut in the web-plates of the ribs near their lower edges. The diagonals connecting the top and bottom members meet at the centre of the latter between each pair of ribs, as shown in the transverse section. This system, whilst it prevents lateral movement of the ribs, aids also in distributing the vertical loads from one rib to another.

The posts, carrying the upper girders of the superstructure, are riveted to the top flanges of the arched ribs and consist of three channel-irons riveted together.

There is no longitudinal bracing between the posts on the inner ribs, but transversely the bridge is braced in the line of the posts by a double set of flat diagonal bars.

The cross-girders rest upon, and are riveted to, the tops of the posts, and are continuous from side to side of the bridge.

The longitudinal girders are fitted between the cross-girders, and are in line with the arched ribs below. Two intermediate cross-girders between the longitudinals further subdivide the spaces between the main cross girders. Flat floor-plates $\frac{11}{32}$ inch in thickness are laid over the whole surface (except across the tops of the piers), and are riveted to the angle-bars forming the top flanges of the girders.

The longitudinals are of the same depth as the cross-girders, and are connected to them by vertical angle-bars at each end of the web-plate; and strips, connecting the top flanges of the longitudinals, are laid across the cross-girders above the floor-plates; the longitudinals may therefore be considered continuous. The floor-plates, riveted to their top flanges and to those of the cross girders, form an additional connection between the whole of the upper girders, both in a longitudinal and in a transverse direction.

The longitudinal girders and floor of the bridge were made continuous, to counteract the effect of unequal loads of the various spans on the piers and foundations.

The continuity is unbroken throughout the length of the bridge, for, between the box cross girders on each side of the piers, girders are placed in line with the rest of the longitudinals, but of larger sectional area than the ordinary longitudinals, so as to compensate for the absence of the additional strength given to the latter by the floor-plates. The connections of these pier girders to the box cross girders are also of increased strength.

Owing to the absence of bracings in a longitudinal direction between the posts, any tendency of the latter to move longitudinally under the influence of a travelling load is checked by the continuity of the longitudinal girders, upon which a pull or a push is thrown by the action of the travelling load.

In the design, the stress that might be thrown on any of the longitudinal girders by a train getting off the line was especially considered. To guard against this contingency also, the longitudinal girders, upon which the lattice-work parapets are fixed, were made of considerable strength.

The ultimate pull of the longitudinals is taken at each abutment by ties securely anchored, and of strength sufficient to control the contraction due to change of temperature. The riveting up of the longitudinals to the cross-girders was done at a temperature of 50° Fahrenheit.

Besides the ties to control the contraction, the abutments are made of sufficient stability to withstand the thrust due to any probable rise of temperature above 50°. On the Middlesex side the thrust is taken by the abutment, and distributed thence on to the viaduct, which extends from the abutment up to the new station building. The pull is taken by tie-bars placed across the top of the abutment, and connecting the longitudinals of the bridge with the continuous wrought-iron girders which form the superstructure of the viaduct.

On the Surrey side, heavy arched ribs of brickwork were thrown across between the back of the abutment and the two adjoining piers inside the Blackfriars goods depot. The piers were further strengthened by buttresses and cross walls of brickwork. By these means the thrust from the longitudinal girders is communicated through the brick arches and into the piers, where its line of direction falls well within the base of the foundations. The pull is taken by tie-bars which pass on both sides of the arched ribs of brickwork, and are firmly keyed into castings bedded against the back of the pier.

With regard to the actual execution of the riveting between the longitudinal and cross-girders, it was only at the joints next the piers and abutments that consideration had to be given to the temperature. All the other joints were riveted as the work progressed, and without reference to the temperature; for the spring of the posts gave sufficient play for expansion and contraction.

When all the joints were connected, except those next each pier and abutment, the lengths for the longitudinals to complete the work were measured when the thermometer stood at 50° Fahrenheit. The girders were prepared, and one end of each was connected to the cross-girder next each of the piers and abutments. The holes for the rivets connecting their other ends to the adjoining cross-girders were then drilled, at the same temperature of 50° Fahrenheit, or as nearly as possible thereto—a variation of about 2° on either side of 50° being allowed. To give room for expansion during the interval before finally riveting these joints, a space was left between the end of each longitudinal and the cross-girder, and this space was fitted at the correct temperature with a packing-plate, the plate being removed for the time, and only inserted when all the joints were drilled and ready for riveting.

At the beginning of January 1886, all the joints being prepared, the packing-plates were inserted when the temperature was low (at which time the contraction of the ironwork gave space for their insertion without difficulty), and the riveting had then to be deferred until the temperature had risen sufficiently to enable the rivets to be got in. During the cold weather, when the thermometer stood at about 26° Fahrenheit, the rivet-holes in the angle-bar lugs, fastened to the cross-girders and connecting them with the longitudinals, did not correspond with the holes in the web-plates of the longitudinals by about $\frac{3}{16}$ inch, thus showing a contraction of twice this amount for each span, and ten times the amount, or nearly 2 inches for the whole bridge, which measures 987 feet between the abutments. As the holes corresponded with one another when they were drilled at a temperature of 50°, this contraction of 2 inches was caused by a fall of temperature of 24°, and this is nearly the calculated amount by which a length of 987 feet of iron should contract.

At the end of January some warmer weather was experienced, and the rivets connecting the longitudinals to the cross-girders were all got in during a single day. The minimum night temperature was fortunately only 8° below the maximum during the day, so that little strain was put upon the rivets by the contraction during the night; the next day the floor-plates over the joints were laid down, thus completing the connections and rendering the whole secure.

Since the final riveting was completed, a large variation of temperature has been experienced; on examination of the rivets no damage appears to have occurred to any of them from the tensional strain to which they have been exposed by the contraction of the ironwork; whilst the abutments show no signs of any movement under the thrust thrown upon them by the expansion.

The whole of the ironwork in the bridge was constructed by the Thames Ironworks and Ship Building Company of Blackwall.

The Board of Trade inspection of the bridge was made by General Hutchinson during the last week of April 1886.

The lines of way on each span were loaded, two at a time, with locomotives extending the length of a complete span. The maximum deflection at the crown of the arch when fully loaded was ¾ inch. The weight of the locomotives was equivalent to 1·5 ton per lineal foot on each line of way, and the proportion of this carried by each of the arched ribs was 0·7 ton per lineal foot. The dead load carried by each rib, including the weight of the rib itself, averaged 0·5 ton per lineal foot; so that the total load on each rib, when fully loaded, amounted to 1·2 ton per lineal foot.

It may be observed that, before any of the superstructure of the bridge was erected, some experiments were carried out at the works of the Thames Ironworks Company at Blackwall, and the load then placed upon the rib amounted to 1·25 ton per lineal foot. The deflection under this load was 0·9 inch; so that the result of this experimental test and that of the Board of Trade agree well with one another.

The load upon the foundations of the piers amounts to 4¾ tons per square foot with the maximum possible load upon the bridge. There was no settlement under the Board of Trade tests.

The total cost of the bridge, including all temporary works, amounted as nearly as possible to £3 per superficial foot of the area of the flooring.

The joint Engineers for the works were Mr. William Mills of the London, Chatham and Dover Company, and Messrs. J. Wolfe

Barry and H. M. Brunel. The chief direction of the design and execution rested with the two last-named gentlemen. The Author of this Paper was the Resident Engineer.

Nine sheets of detailed drawings are submitted with the Paper, and from these a selection has been made, and reproduced in Plate 5.

Discussion.

ttwell. Mr. G. E. W. CRUTTWELL said that one of the chief points of interest in connection with Blackfriars Bridge lay in the methods adopted in the erection of the main ribs of the superstructure. The only stagings used for this purpose, besides those immediately surrounding the piers, consisted of two rows of piles driven on each side of the water-way of each span, and supporting platforms placed just beneath the soffits of the arches. The ribs were brought up from the works at Blackwall in three pieces. The middle piece was of such a length as to span the water-way between the two stages, and to take a bearing upon the platforms on either side. The middle piece of the rib weighed 16 tons, and the two end pieces 11 tons each. About slack-water of high-tide the barge containing the rib was swung across between the two stages, and the middle piece was lifted out by two cranes, placed on each side of the water-way, and was put in position on the platforms. The two end pieces were each lifted out by a single crane and lowered down upon the middle piece. There were thus two joints to be riveted in each rib. The rivets composing the joints amounted to 9½ per cent. of the total number in the complete rib. He had placed on the table some photographs showing the ribs in the course of erection. After the ribs had been placed in position half-timbers were slung beneath their undersides, to carry the planking laid between the ribs to prevent the fall of rivets, &c., into the river. Those half-timbers formed the only diminution to the permanent headway beneath the ribs during the course of construction. In each span a constant water-way of 68 feet was maintained between the stages. The Thames Conservancy had agreed to allow the river to be blocked twenty minutes whilst the middle piece of the rib was being lifted. The first rib took twenty-five minutes to lift, but afterwards, when more practice had been acquired, fifteen minutes were found to be sufficient, from the time of swinging the barge across between the two platforms to the time of placing the middle piece of the rib in position on the platforms. The total weight of iron in the bridge amounted to just over 5,800 tons. Originally it had been intended to have four lines only; but it was afterwards decided to add three additional lines, making seven in all. The works were commenced in February 1883. The order for widening

Mr. Cru

was only given in January, 1884, and as the bridge was opened in May 1866, he thought the Contractors, Messrs. Lucas and Aird, were to be complimented on the rapid and able manner in which they had executed the work. The contractors were represented by Mr. Henry Turner, who, together with Mr. Hodson, Engineer to the Thames Ironworks (who manufactured the whole of the ironwork in the bridge), arranged the method of erection.

Sir Brad Leslie.

Sir **Bradford Leslie**, K.C.I.E., said he had read the Papers with great interest, and he thought them very important, as affording the latest information with regard to the subject of foundations sunk by open excavation. There were many cases in alluvial deltas of large rivers, where it was necessary to put down foundations for heavy works—bridges, quay-walls, docks, warehouses, and other structures; so that the question of foundations in soft alluvial soils was a very important one. The system of securing a good foundation in alluvial deposits by sinking wells, caissons, or rectangular blocks of masonry by excavating from the interior, if it did not originate in India, had at least been more extensively practised there than elsewhere. It was applicable to foundations of any shape and any size, indeed the larger the area of the block to be sunk the more easy it was to keep it straight, and the less likely it was to be hung by side friction. It was generally preferable to use one large caisson or block rather than several smaller ones. Where several caissons were sunk in close proximity, one sometimes disturbed the others, and if they had to be sunk to any considerable depth they were apt to jam one another. In the case of Blackfriars Bridge, there were no doubt good reasons for using three separate caissons for each pier; but for a very deep foundation a single caisson with three excavating chambers would have been preferable. With a single caisson 90 feet long there would have been no risk of tilting up or down stream, and it was probable that lateral canting would have been more easily controlled. All such caissons or block foundations in India, whether cylindrical, oval, or rectangular in plan, were made with a vertical external surface, experience having shown that the vertical sides acted as a guide for the caisson and disturbed the ground less than any other form; the difficulty met with in keeping the caissons of the Hawkesbury Bridge straight was conclusive on this point. There was one other point, however, which might have been overlooked at the Hawkesbury Bridge, and attention to which might have prevented some of the trouble that occurred with the caissons there. He did not know what the "mud" at the Hawkesbury Bridge was like—it was rather an indefinite radford expression—he supposed it was more or less permeable to water, and in sinking through such material he had found it desirable to avoid any considerable depression of the water inside the caisson. The dredgers commonly used baled out a quantity of water at each lift faster than the water could percolate in through the bottom, and by degrees reduced the level of the water inside the caisson considerably below the level of the water outside, and this difference of level might be increased by the incoming tide. When the difference became very great, it caused a blow-in below the cutting edge of the caisson, and a large quantity of mud or silt was carried in from outside, leaving a cavity outside towards which the caisson tended to travel. The stuff carried in was not always from below the caisson but from the side, and when once such a weak place was established, it had a tendency to increase with each succeeding blow-in. He had no means of knowing that anything of the sort took place at the Hawkesbury Bridge, but it had given much trouble elsewhere, and the large quantity of stuff carried in with the blow had all to be excavated before further sinking could be effected. The best plan of avoiding such blows was to keep the water inside the caisson at all times level, or nearly so, with that outside, and to depend upon dead-weight for sinking. In that way the caisson went down gradually, and the quantity of stuff excavated was limited to the net amount displaced by the caisson. It was remarkable that when there had been a great difference in level between the water inside the caisson and the external water, and a sudden blow-in occurred, the water in the caisson generally rose considerably above the level of the water outside, and even overflowed the top of the caisson, and this, he believed, was what caused the damage to the No. 4 pier of the Benares Bridge. If, during dredging, the water in the caisson had by means of siphons or pumps been maintained at the same level as the water in the river, probably this disaster would have been avoided. The skilful manner in which the breach in the caisson was repaired by Mr. Walton deserved the highest praise. He remembered that at the time there was some talk of abandoning the No. 4 pier, and altering the spans of the bridge by sinking two caissons on either side of it, and a less able engineer than Mr. Walton might probably have been glad to adopt the alternative. In sinking through clay, or any material more or less water-tight, the lowering of the water-level in the caisson rendered the portion of its weight that would otherwise be water-borne effective for overcoming side friction, and greatly facilitated sinking; but any

Sir Brad Leslie.

considerable lowering of the water-level in permeable strata was at the risk of sudden irruptions of mud and water from outside, causing the caisson to tilt and travel laterally. The steel caissons of the Hawkesbury Bridge appeared to have extended, with dredging-tubes, complete up to the level of low-water, and this was the case with the caissons of the "Jubilee" Bridge. At the Benares Bridge the iron caissons were of various heights, according to the depth of water in which they were to be pitched. Above the iron caissons, and attached to them, a framework or "crating" of angle-bars was built, which connected the brickwork of the caisson above the iron shell with that below, and this system of construction must have resulted in a great saving of outlay as compared with complete iron or steel caissons like those used in the Jubilee or the Hawkesbury Bridge. The accident, however, to No. 4 pier of the Benares Bridge showed the danger of dispensing altogether with the iron shell. Considering the conditions to which such foundations sunk by dredging were subjected, it might be found expedient to adopt a form of caisson differing from any of those used at the Hawkesbury, Benares, or Jubilee bridges. For floating into position, pitching, and withstanding the varying strains of sinking, an iron or steel caisson, with internal bevelled surfaces and dredging cylinders, equal in depth to not less than one-fifth the greatest length in the plan, was indispensable. Above that level it would appear sufficient to carry up the metal dredging-cylinders, connecting them together at intervals, and with the external brickwork, by suitable bar-iron bracing. The building of the external brickwork in cement would go on simultaneously with the dredging, and would cause no delay, and no doubt it would be more solid than concrete thrown in among a network of stays and struts in which it was hardly possible to avoid cavities. As compared with the Hawkesbury and Jubilee Bridge caissons, such a caisson would show a considerable saving, while the continuous iron or steel dredging cylinders would protect it from such an accident as that which occurred to No. 4 pier of the Benares Bridge. The weight on the foundations of No. 6 pier of the Hawkesbury Bridge was stated to be 9 tons to the square foot. This, it was presumed, was without allowing for any support from side friction, which, considering the unstable nature of the mud, could not amount to much. The area of the 2-foot splay at the base of the caissons, 245 square feet, must, however, carry the weight of the superincumbent mud and water, equal to 7 tons to the square foot, or 1,715 tons on 245 square feet; this increased the average pressure on the total area of the base, 1,134 square idford feet by 1½ ton to the square foot, making it 10½ tons altogether. At the Benares Bridge the weight on the base of the piers was 11·19 tons to the square foot; at the Jubilee Bridge, disregarding side friction, it was 9 tons to the square foot, and at the Gorai Bridge it was 8½ tons. Such loads were sometimes objected to as excessive, and compared with the 4¾ tons to the square foot on the foundations of the Blackfriars New Railway Bridge, they appeared to be so; but it was simply a matter of depth. If the foundations of the latter bridge had been 100 feet deeper, the weight on the foundations would have been increased to 9¾ tons to the square foot at least. As exemplified in the case of the Hawkesbury Bridge very little advantage was gained by splaying out the base of a deep foundation, because the area so gained had to support the normal weight of the superincumbent earth and water. The methods adopted for the erection of the superstructures of the Hawkesbury and Benares bridges, were in each case so admirably adapted to the circumstances, and were happily so successful, that it was not easy to suggest any modification which would have been an improvement. It would be interesting to learn the weight of a main span of the Hawkesbury Bridge as launched, and as subsequently completed.

itheson. Mr. EWING MATHIESON said that in the case of the Benares and the Hawkesbury bridges, although the superstructure and the large spans presented very much of interest to engineers, he thought it was in the sinking of the foundations that the most interest would be felt, and that there was most to be learned. In the case of the Hawkesbury Bridge, there was apparently a very heavy weight upon the foundations. The Author had stated that a weight of upwards of 9 tons per square foot was imposed on the base of the cylinders. There was no mention whether that included side pressure due to the wind, nor was there any statement as to any of it being relieved by the friction of the piers. As the piers were splayed out at the bottom, perhaps it would not be fair to take into account much gain from the friction; but considering that the material on which the bridge rested was only a kind of mud, although no doubt it was greatly compressed, 9 tons weight seemed to be rather considerable. In the case of the Benares Bridge 11 tons were imposed in the gross; but the Author estimated that that might be reduced by one-half, because of the amount due to the friction in the piers, and to the displacement. Still, taking one-half of 11 tons on sand, it appeared to be pretty considerable when it was remembered that on the London clay in the Thames, which was one of the best and easiest foundations in the world,

Mr. Math

not more than 6 tons had been imposed in the case of any of the big railway bridges crossing the river. The Hawkesbury Bridge was stated to have been built by American contractors. It appeared to him very likely, although no mention was made of it, that the American constructors had information that was not available in England in regard to the exact nature of the soil. If only occasional borings were taken, and it was only known generally that the foundations were mud, bridge constructors would feel rather shy of undertaking all the responsibility, unless there was some margin to meet the case of unexpected events. It was interesting to note, both in the Hawkesbury and the Benares bridges, that a depth beyond that available by pneumatic plant had been reached. He did not know of any case where more than 120 feet had been reached by such methods. Now, while it was comparatively easy to excavate by mechanical means in the open at that depth, and even at a greater depth, great difficulty would be experienced if trees or boulders or any unexpected obstruction occurred, because some new method, of which he believed no example was as yet known to bridge engineers, would have to be found for dealing with them. Either, then, the American engineering contractor was more daring or more speculative than English constructors, or he had some information not available in England with regard to the stratum. Probably some power on the part of the engineer to grant a higher price if obstructions were met with would meet the case. In the case of the proposed Channel Bridge, also, it must have been assumed that no obstructions would be met with, since, as the depths were too great for air caissons, the excavations would have to be performed entirely by mechanical means from the top, no attempt being made to remove the water in the cylinders. But even there, although the foundations were of chalk, 9 tons, he believed, was the maximum loading, including even wind-pressure, and that was deemed excessive by some engineers who had criticized the design. The drawing of the Benares Bridge stopped at the abutments. The Paper stated that the shore was liable to be inundated for some miles inland in rainy seasons. It would be interesting to know how the approaches were built, whether on embankment or bridge arches. Cases, he thought, sometimes occurred where, although the approaches were apparently on dry land, it would be cheaper to put down cylindrical piers, sunk as those in the river, than to build them of brickwork; no doubt, however, a great deal depended upon the comparative local cost of the different materials.

Maynard. judging from the description given of the Benares Bridge, there was not half the width of base in Hawkesbury Bridge, but there was about the same height. When he said half the width of base, he meant measuring it from such a point as would fairly represent the effective width. If the piers were solid, and on unyielding ground, then, of course, the difference would be much more in its favour; but, in consequence of its not going down to very hard rock, there was a greater tendency to yield to the wind than would otherwise be the case. Although friction in going into the mud might at one time prevent the caissons from sinking, at another time, as he had often seen, when the tide rose, the ground all round would become so saturated as to be little more than slurry, so that unless the piers were really carried down to the solid rock, there was no certainty about their having been founded in the best possible manner. He had frequently noticed, not only on a rising tide, but also in a country where the ground had been saturated by the rainy season, the same kind of thing happening; at a very great depth, the ground became so saturated that it might yield to the pressure. Although the bridge now stood, in a very short time the condition might be altered, not because the design was wrong, but because the pier had not reached the solid rock. It might be a very serious matter for consideration at some time or other. On the whole, he thought that the bridge had been very cheaply and quickly made; it had done credit to those who had erected it, and they had escaped without any disaster. The whole erection seemed to him to have been a series of very considerable risks, and ought to cause a little reflection. The splaying out of the caissons was a matter which had been over and over again thought of, and it had been seen by the experience of former years that it was useless to expect a bell-shaped caisson to go down straight without anything to guide it. It was always desirable to have vertical sides, or something representing them. That was found to be the case, and it answered very well, in the Dufferin Bridge; but, in the erection of the Hawkesbury Bridge, there was a gaining of experience which had already been acquired by the sinking of caissons.

Mr. W. Martineau wished to mention a somewhat unusual circumstance that had occurred to him in the sinking of cylinders for a bridge in Brazil, as it might be useful to those engaged in similar operations. The cylinders were only 4 feet 6 inches in diameter, and they were sunk by divers in the first instance through sand and loose stones, which were the remains of an old bridge that had occupied the same place. They were sunk to a bed of impermeable white clay, and when they were well into it they were pumped perfectly dry, and the remainder of the work, adjusting the cylinder to its exact depth, was carried on digging from inside the cylinder. Three pairs of cylinders were sunk with perfect success. The seventh had gone to a depth of about 40 feet, and was in the clay. It had stood for some hours and was pumped perfectly dry, and was holding 40 feet head of water as well as the other cylinders had done. A man went down to complete the sinking by digging out the little remainder of the clay, and on reaching the bottom he fell insensible. His companion went down after and did the same. They were then obliged to get a diver, who was fortunately on the spot, to put on his helmet and go down after the men. He did so, and brought them up, but unfortunately they both died. It was evident that the clay at that point gave out carbonic acid gas, no sign of which had been met with elsewhere. It was a very unexpected accident, and one which should be borne in mind in carrying out similar works. The danger existed especially in the case of rivers in tropical climates. The bridge to which he had referred was in an estuary of a river in Brazil, where he presumed, from the decayed vegetation and peat above and the pressure, the gas had been generated, and discharged itself into the cylinder at that particular moment. Mr. Martine.

Sir JAMES N. DOUGLASS said that, as the discussion had rather turned on the question of sinking cylinders, he desired to offer a few remarks. The outward splay of the bottom of the Hawkesbury Bridge cylinders was doubtless the cause of much of the trouble of sinking, as stated by the Author at p. 7, in consequence of the tendency of the cylinders to deviate from the required direction. After many years' experience in sinking parallel cylinders, he might state that he had never experienced difficulty from this cause. He would refer to two instances of sinking small iron cylinders through soft ground to considerable depths about forty years ago; in both cases the cylinders were parallel throughout, with a chisel edge at bottom. The first case was through the alluvial soil of the foreshore of the Thames at Sea Reach and near low-water of spring-tides. The structure was an iron pile lighthouse on seven 18-inch cylinders, one central and six surrounding it, all sunk perpendicularly. It was found that when the cylinders had been carefully guided to a depth of 10 to 12 feet they invariably went down truly without further care. They were sunk by misering tools, and were carried down through mud, loam, and sand to compact shingle at a depth of 56 feet. The other case was at the Sir James N. Douglass.

Sir James N. Douglass. Goodwin Sands, and about the same date, where a single cylinder 2½ feet in diameter was sunk in the same manner, by misering tools, through the sand to the solid chalk, a depth of 75 feet. A staging was formed on the sand with short iron piles, and the cylinder was sunk perpendicular from its centre. The various strata of material sunk through were nearly:—

Surface to 20 feet. Clean, sharp, coarse sand.
20 to 40 feet. Ditto, slightly tinged with sedimentary matter. With microscope specks of comminuted shells perceptible.
40 to 45 feet. Ditto, deeply stained and interspersed with small lumps of blue fœtid clay.
45 to 50 feet. Perforated chalk, lumps of extinct pholas cells, wormed stones, decayed mud, broken shells, and small beach shingle, dark silt and sand.
50 to 55 feet. Clean sand.
55 to 60 feet. Clean bright sand with broken shells.
60 to 65 feet. Sand deeply tinged with sedimentary matter, with gravel, small fragments of decayed timber, and pieces of coal.
65 to 70 feet. Sand very black and fœtid, with lumps of clay.
70 to 75 feet. Shells, small chalk nodules, and rounded pebbles.
At 75 feet. Solid chalk with a slight covering of mud.

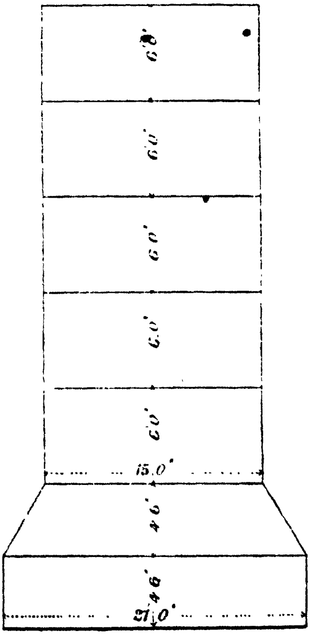
He had always found, in going through submerged sand that was at all clean and free for the percolation of water, the cylinder was aided in sinking by keeping the surface of the water inside the cylinder rather lower than the surface of the water outside. If the inside water was too much lowered a run would occasionally occur, especially with flood tide. He had known a cylinder suddenly sink 8 to 10 feet.

Mr. Berkley. Mr. G. BERKLEY, Vice President, said that he knew something of the plans of the bridges described, having occupied a very agreeable position with Sir Douglas Galton and Mr. Barlow in forming a judgment on some sixteen or twenty plans sent in for the construction of the Hawkesbury Bridge. Of course, he would have preferred that the work should have been executed by an English firm, but the gentlemen who had worked so satisfactorily in America might be congratulated on taking the position which they had done. The Hawkesbury and Dufferin bridges were really grand works. What would have been thought a few years ago of bridges of 415-feet span going down for their foundations 170 feet? He thought the profession might be proud of those works having been so successfully carried out under the disadvan-

Berkle tages which would always arise far from the centre where works and the men who carried them out were found. It required perhaps more courage to give a description of difficulties than successes. There had been no shirking, but a full description had been given of the mishaps that had arisen. Mr. Maynard had expressed some doubts as to the stability of the foundation, and had given as his reason some statements which appeared to be erroneous. He had stated that the depth of the foundation originally anticipated was 170 feet, and that the principal material to be passed through was mud. He added that the deepest foundation was only 147 feet, that it had probably rested on mud, and might fail. The fact was that the deepest foundation was 162 feet. It was stated in the Paper, "The caisson was then sunk through the mud;" which would lead to the belief that it had gone through the mud, and had got on to the solid sand, which he understood was below the mud. There was therefore no occasion to fear the failure anticipated by Mr. Maynard.

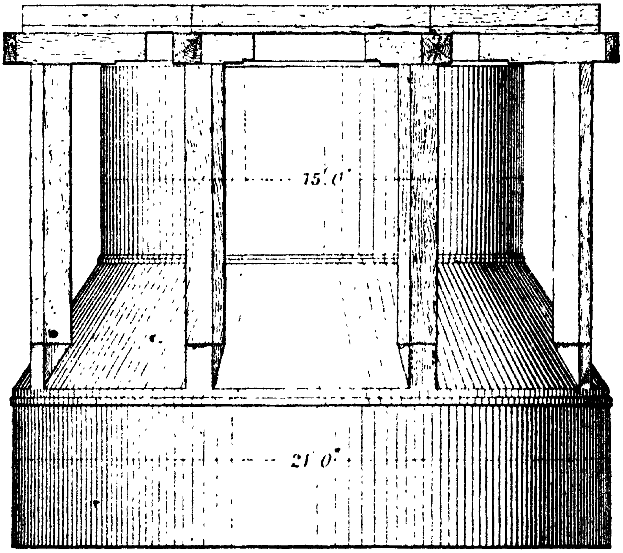
Mr. Walmisley. Mr. A. T. Walmisley thought it might be interesting to put on record some account of the cylinders of the Albert Bridge over the Thames at Chelsea, not because of the depth they went to, but because of their being cast in complete rings. The foundation or lower cylinder of the Albert Bridge was 21 feet in diameter and was cast in a single ring 4 feet 6 inches deep (*Fig. 2*). Upon that came a conical ring which enabled the diameter to taper from 21 feet to 15 feet; then cylinders 15 feet in diameter were carried up each 6 feet high, and bolted together horizontally with internal flanges in the usual way (*Fig. 3*). There were no vertical joints in those cylinders. They were sunk by having first, outside guiding piles for the 21 feet cylinder at the bottom; then triangular pockets were cast on to the sloping cylinder, which tapered from 15 feet to 21 feet in diameter, and the triangular pockets held the interior guiding piles for sink-

Fig. 2.



Walmisley. ing all the upper rings 15 feet in diameter on the top of the conical ring. He was pleased to see that the engineers for the new Blackfriars Bridge had adopted a continuous pier, because in cases where there were several lines of railway crossing the bridge, it was difficult to determine exactly where the weights might produce pressures, and hence a continuous pier was preferable to an isolated pier such as would be provided by cylinders. He was also pleased to see that a curve of a proper shape, to suit the stresses in an iron arch had been adopted, which was not the case in some other well-known arched bridges.

Fig. 3.



Sir Benjamin Baker. Sir BENJAMIN BAKER, K.C.M.G., said that he had promised if Mr. Burge would write a Paper on Hawkesbury Bridge, he would defend it at the Institution. He was pretty familiar with the bridge because he reported to Mr. Whitton, the Engineer-in-Chief of the New South Wales Government about eleven years ago, and said that there would be no difficulty in building a bridge across the river at that point, and that the best plan would probably be that of open-top caissons, sunk by dredging, and a steel superstructure floated out into position. His estimate for the cost was about £400,000. Nothing was done for some time because, as Mr. Berkley had stated, it was thought to be a rather formidable thing to sink piers to the depth then indicated. At length invitations

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to tender were issued, not only in England but in America, and on the Continent, and a number of very interesting designs were sent in. The design recommended by the consultative body was that which had been carried out by the Union Bridge Company, one of the most experienced bridge-building firms in America. The question of the piers was naturally the one first discussed, and the members of that body selected the form of caisson which was adopted with their eyes entirely open. He had told them of the experience in England, but they had their own experience in America, and they thought that on the whole, as there was some uncertainty about the character of the alluvial deposit, whether it would be close sand which would grip the caisson as if it were in a vice and not allow it to go down at all, or whether it would be mud, the method they adopted of giving a slight batter to the sides, gave them the least risk of the caisson sticking or getting out of place. As a matter of fact two of the caissons had got somewhat out of place, but there was really no grounds for Mr. Maynard's alarm as to the final stability of the piers. He had received a letter on the subject, which was dated June 1887. When the No. 5 caisson was very badly out of place, it was sunk down to a depth of about 100 feet from the water-surface, and was overhanging about 10 feet at the top. After a few feet further sinking the caisson became perfectly vertical, the reason being that it had got into relatively hard ground. The report from the Union Bridge Company at the date he referred to was "You will see that as soon as we got into the black sand, we were able to level up the caisson, and as she has some 32 feet odd to go, we hope by inclining the bottom the other way to get her back into position, as the sand will hold the bottom firmly enough." So that instead of resting on the mud, as Mr. Maynard had suggested, there was what was called black sand, for a thickness of 32 feet, before getting to the hard ground upon which the caisson finally rested. With reference to No. 6 caisson, it twisted in a very peculiar way; whatever was done in the way of dredging it persisted in rotating, and it got out of position about 5 feet. The only reason that could be assigned for it was that one corner of the caisson got into hard sand rather quicker than the other, and so the caisson began to rotate. Of course, if the caisson had been perfectly circular, as in the design submitted by a French firm for the same bridge, it might have gone on rotating with impunity. Mr. Barlow had suggested that the piers might have been weighted before erecting the girders. It was a very formidable and costly thing to weight piers which were to carry such a heavy load, and a

Sir Benjamin Baker. little settlement really did not much matter. In the case of the Hawkesbury Bridge the piers had leant a little over, but he did not think the settlement was more than ½ inch, which was quite immaterial, being nothing but what happened to other bridges. At the same time as the Union Bridge Company was building the Hawkesbury bridge it was making another very large one across the Hudson about 80 miles above New York, the Poughkeepsie Bridge, through similar material and with foundations nearly as deep. In that case there was no trouble in getting the caissons vertically down. They were built of timber and they had a batter of about 1 in 60 or 1 in 80 in the sides, and instead of three dredging tubes in the centre they had two rows, of six each, or twelve in all, square chambers, in any of which the dredger could be worked. That, he thought, would meet the views of Mr. Burge. One side or the other could be manipulated by dredging and so the caissons be kept vertical. There had been a little settlement he believed in this case as at Hawkesbury, but immaterial in amount. Although the Hawkesbury Bridge had been taken very pluckily by the Union Bridge Company as a guaranteed contract and at a reasonable price, without putting on an enormous sum for contingencies, as many English makers did if they had to run the risk of erection abroad, yet on obtaining the contract the company had no national prejudices about how it should be carried out, and sublet nearly the whole of it. The superstructure was sublet to Messrs. W. Arrol of Glasgow and the caissons to Messrs. Head and Wrightson. Then the sinking of the piers was sublet to Anderson and Barr, a Dutchman and an American, and the masonry to Mr. Samuelson, contractor, of New South Wales. The erection of the bridge was also, he believed, partly sublet to someone else, so there was very little detailed work left for the original firm to do. The company made the design, however, and had to take the whole control and pull it all through. He thought it deserved credit for doing so—credit in the first instance, and equal credit for sub-dividing the work afterwards because someone else could do it cheaper. No doubt it could have done the work alone, for the company had first-class works in America where it could turn out 20,000 tons of girder-work in a year and was an experienced bridge erector; it was therefore not from incapacity, but simply because it could get the work done in a better market equally well and cheaper. In the same way he thought the New South Wales Government and its consulting engineers deserved credit in not allowing national prejudices to interfere in any way in awarding the original

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contract to an American company. The superstructure, as he had said, was made at Glasgow—everything except the forging of the eye-bars. He had exhibited a piece of an eye-bar which was rolled at Glasgow. It was part of a long steel bar, 8 inches wide and 2 inches thick, and had been bent cold under the steam-hammer, and every bit of the steel was equally good. The test that he had seen carried out in America on a complete eye-bar gave 29½ tons per square inch over the whole section, an elastic limit of 16 tons, and an elongation of 37 per cent. in a length of 1 foot, and 25 per cent. over the whole length. It seemed a very strange thing that steel bars made in Glasgow, and tested there, should then be sent to New York, and 400 miles by rail inland in order to be jumped up at the ends and have two eyes formed. Why could not the whole process be done in Glasgow? It was as simple as possible. There was a rough and ready machinery at the Union Company's works to do it, which was nothing to the kind of plant to which engineers were accustomed in Great Britain. A furnace to heat the bars, a simple steam-plunger machine to upset the ends, and a light steam hammer and dies to finish the eyes, were all that there was at Buffalo. The ends of six bars were put in a furnace, and were heated to a forging heat for a length of about 4 feet. Each of the bars was taken out successively and slipped into a machine where there was a sort of toggle-grip which would hold the bar firmly by the edges for a plunger to come and upset the end of the bar into a die formed by a fixed bottom mould shaped to the width and rear curves of the eye, and a top movable die held down by steam-pressure. The rammer was pushed forward, not by a hydraulic cylinder, but a steam cylinder 66 inches in diameter, with 120 lbs. steam. The head of the bars was 18 inches wide, whilst the body was 8 inches, so that the bar had to be upset to an extent of 10 inches. The pressure of the steam would give a squeeze of only 180 tons, which was a mere nothing, only about four times as much as was used for closing an inch rivet. With that force the bar widened out as easily as possible to 18 inches. The success depended upon some little refinements. Great difficulty had been experienced in trying to do it in England, some time ago, when hydraulic pressure of 400 or 500 tons failed to form the eyes of the new Hammersmith Bridge links. He quite understood how the difficulties had arisen. Facilities in the die must be given for the bar not only to increase in width, but also to thicken. At the first squeeze it got from the plunger it thickened about ¼ inch, and the top die which was held down by a steam-cylinder also yielded

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about ½ inch; then the second shove that it got was that of a circular die, which gave it the form of the head; that thickened it another ¼ inch. The eye was thus formed 2¾ inches thick and about 18 inches wide on a bar originally 2 inches by 8 inches. It then went from the plunger to the steam-hammer, and was simply hammered down in the die to the shape of the head, a sort of button being put at the bottom of the die which began to punch the hole. Then it was heated again, and reversed, hammered, and swaged, and the complete eye was formed with the hole through it ready for boring; it did not want any planing or trimming at all, but was completely finished. That one machine turned out, he believed, forty eye-bars a day, working double shift, and as far as he could make out the cost was only about £2 a ton. Reverting for a moment to the piers, he might say that it was a mistake to assume that caissons or cylinders with parallel sides would go down necessarily vertically. Two years ago, when at Chicago, he was consulted by the contractor who was doing the Lake Tunnel. He made two attempts to get down the shaft for the tunnel; one shaft he abandoned altogether, and then he started another, and one day, although it was a perfectly vertical-sided circular shaft, it went 6 feet out of the vertical, owing to some general movement in the soft soil through which it was sunk. The shaft was not sunk in the water, it was as if it had been sunk in the middle of the road; and yet, owing to the disturbance of the ground from taking a certain amount of weight off the soil locally, it had been drawn from one side, and the cylinder was put out of the vertical. He had not much to say with regard to the other bridges except in the way of approval. He remembered seeing the caissons of the Dufferin Bridge at McLellan's Works at Glasgow. They looked very formidable structures, standing up against the sky-line, and he was very glad to find hey had been successfully sunk. With regard to the foundations of the Blackfriars and other bridges in this country, he had often wondered why English contractors did not adopt the plan which ontinental contractors almost invariably followed in a case of hat kind—having the caisson in one piece, and putting it down by air-pressure. In that way they could, as a rule, get in their oundations without anxiety or risk in less than half the time, and t less cost, than with dredging and divers' work.

Mr. Matheson.

Mr. EWING MATHESON asked permission to say a word with reference to Sir Benjamin Baker's remarks on the manufacture of links, and he would ask Mr. Berkley to confirm his statement. It was easy enough to make the links, if engineers would only use them. In America there were tens of thousands used every year, and bridge builders had an ingenious apparatus for making them. It was not simply a question of the cost of doing the work, but a very expensive plant was required. Twenty-five years ago some 30,000 tons of similar links were sent from England, chiefly by Mr. Berkley. They were made by a firm no longer existing, Messrs. Howard and Ravenhill, from high quality scrap-iron. They were made in a manner superior to that usual in America, the link being rolled in one piece without upsetting, an operation which, to a certain extent, altered the fibre. If a few thousand tons of links were wanted, the machinery would soon be made for the purpose. Mr. Matheson.

Mr. Davis. Mr. ALFRED DAVIS observed that Mr. Matheson was not quite correct in stating that upsetting altered the fibre. He had a photograph of one of the links manufactured in America which had been planed and exposed to the action of acid, showing that the fibre ran round the end of the link. As to the cost of machinery for manufacturing, it was a very simple contrivance, consisting as Sir Benjamin Baker had described it, of a steam-cylinder with a piston pressing against the end of the bar and upsetting it. After upsetting, a hole was punched in the head, which being reheated was placed in a die, corresponding to the exact shape of the head required, with a projection corresponding to the hole previously punched, and the operation completed under a steam-hammer, the pin-holes being subsequently bored out.

Mr. Berkley. Mr. G. BERKLEY said that he had, as stated by Mr. Matheson, sent out a number of links, and they had proved perfectly satisfactory.

Sir Frederick Bramwell. Sir FREDERICK BRAMWELL said that he had seen the upsetting instrument in use in the United States. He also well recollected the method of making the eyes by Messrs. Howard and Ravenhill. That method was introduced in order to get rid of the difficulty that formerly prevailed of welding on the eye, a process which nobody liked. Howard and Ravenhill succeeded in making the eyes out of the solid without upsetting, and did so by a peculiar method of rolling. But there did not appear to be a market in this country for the article. He saw the plan at work in the United States, but on his return he did not feel at liberty to talk about it, because he understood there was an intention to patent it. He had expressed his surprise that it should be publicly and commercially used without being patented, and the reply was that in the United States inventors were allowed to use their invention for a certain length of time, if they announced their intention of

Sir Frederick Bramwell. patenting, and could afterwards take out a patent or not, as they pleased.

Mr. Cowper. Mr. E. A. Cowper said that he could speak to the manufacture of a large number of links for the Kieff Bridge in Russia, by Messrs. Fox and Henderson. A bar was rolled wide at the ends as it was passed sideways through the rolls, according to the plan followed. The body of the bar was then reduced by rolling longitudinally; in that way they got the wide eye at the end. The whole body of the iron was in one piece. As to the moving of a large body of material in the case of foundations already in, he knew of one instance of a wall, 75 feet high, placed by a river, having a cofferdam in front; the piles of the cofferdam were drawn instead of being cut off, and in the night the whole wall and foundations moved 18 inches towards the river, and all cross walls were broken. A friend of his walked through a space of 18 inches in the cross walls. The wall did not go out of the upright to any extent, and the building had stood to the present day unmoved.

Mr. Walton. Mr. F. T. G. Walton remarked, in reply to the discussion, that Sir Bradford Leslie had mentioned the advisability of keeping the water inside a caisson, when in process of sinking, to the same level as the external water. He fully agreed with him on this point which was very important when the caisson was passing through the permeable upper strata. In the case of No. 4 pier of the Dufferin Bridge, however, the internal and the external water were practically at the same level when the accident occurred, which was therefore not attributable to excess of water-pressure externally. It was solely caused by the inrush of silt which overlaid the clay, on which the cutting-edge rested, to a thickness of 49 feet 6 inches. This silt weighed 96 lbs. to the cubic foot, or 34 lbs. per cubic foot heavier than the water inside the caisson. When, therefore, the clay below the cutting-edge gave way, and fell into the excavation made by the diggers, the silt at once rushed in to a height of 30 feet, and displacing the water, caused it to rise 21 feet in the caisson, and to pour over the top in large volume. Soundings had been taken round the pier shortly before the accident, and the fact that the silt rushed in from above the clay, and not from any spring below it, was fully established by the soundings taken immediately after the accident, which showed a large depression in the silt at the upper end of the pier where the blow-in occurred. This point was thus fully gone into because all sorts of theories had been started at the time to account for the accident. The proposal of Sir Bradford Leslie that the metal dredging-cylinders

Mr. Walton.

should be carried up inside the brickwork above the point where the caisson—which was the "well curb"—ends, was excellent, and would render it possible to build up and sink the pier rapidly, without waiting for the masonry to set, and without fear of injury. With reference to Mr. Ewing Matheson's remarks on the weight imposed on the base of the Dufferin Bridge piers when founded on sand, it must be remembered that sand at that depth was really incompressible after the pier was firmly seated on its concrete plug or hearting. After hearting the first deep pier completed, and when building up the superstructure which added about 5,000 tons, the pier foundations settled uniformly about 3 inches. This was due to the concrete hearting not having completely filled the conical portion of the excavating chamber at the base of the caisson, and the pier was consequently free to settle until firmly seated, or until the cavities left in the caisson were completely filled. In hearting the other piers, the plan was adopted of filling in the conical portion of the caisson with clean sand, which would spread in the water and completely fill under the splayed base. The result was most satisfactory, the settlement in the piers thus treated being inconsiderable as compared with the 3 inches first noted. In sinking the piers no large trees were met with; but elephants' remains were found on a layer of gravel at 125 feet below low-water, and it would appear therefore that it was a matter of luck not coming across trees at great depths beyond the reach of a diver. Other serious obstructions were, however, met with in the shape of layers of kunkur rock, varying from 9 inches to 2 feet 6 inches in thickness. These were broken up by working an 18-cwt. chisel in the excavating chambers. The diggers then drew out the sand or gravel below the layers, which gradually crushed in with the weight of the caisson and came out in huge pieces. In a similar way large masses of masonry were torn in pieces and removed from No. 4 pier after the accident. It was difficult to compare the depth of the foundations in the Hawkesbury and Dufferin bridges, because in the former, the depths were given below high-water of spring-tide, the difference between that and low-water not being stated; whereas in the latter the depths were below low-water level. It was the difficulties encountered in carrying out the works on these three bridges which rendered the work most interesting to engineers, but, unfortunately, they could not all be stated. He might, however, mention one difficulty which arose at the time of erecting the main girders of the Dufferin Bridge, and which was at the time a source of extreme anxiety. In span No. 2, the temporary screw-pile pier next to No. 2 main

Mr. Walton. pier, had to be placed in 45 feet depth of water on a hard clay bed of irregular surface into which the piles could not be screwed. The work had to commence before the river had subsided, and as the strong current would not allow of sand being used for flushing up the irregular bed of the river, it was determined to employ rubble kunkur for this purpose. On the top of the kunkur were placed bags of sand to a depth of 6 feet, and on the top of all was formed a mound of sand as described in the Paper. The piles were pitched and screwed down into the layer of sand-bags, the pier was erected, the upper platforms were completed, and the erection of the main girders had commenced, when settlement was observed in the temporary pier. This settlement increased daily until it reached 3 feet, and the pier leaned over considerably upstream. There was then a weight of about 200 tons on the pier, or 16½ tons on each pile. It was evident that some of the piles must have had their bases broken by unequal pressure. The superstructure was at once lightened and the following steps were adopted:—The sandbank round the pier was brought up to a foot above water-level, and four rows of 12-inch wooden piles were driven, eight piles in each row, or thirty-two in all. These piles were driven to an average depth of 30 feet, and so solid was the artificial sandbank that the piles could then be depended on to carry 12 tons each. A timber framing was erected on these piles, with rails on the top so placed under the shoulder-joints of the screw-piles, that steel wedges would throw the entire weight of the structure on to the wooden piles. The superstructure was then packed up, its distortion rectified, and the main girders were erected with no further trouble. In taking out the screw-piles afterwards, it was found that the screw-flanges had been broken off in seven out of twelve. In this case the entire weight of one span of the temporary structure, together with its proportionate weight of the main girders and the travelling cranes, nearly 500 tons in all, was carried on piers resting in made ground deposited under water. This experience seemed well worthy of record. He had omitted to mention in the Paper that the girders were made by the Patent Shaft and Axletree Company, at the Old Park Works, Wednesbury.

Mr. Burge. Mr. C. O. BURGE, in reply to the discussion, remarked that Sir Benjamin Baker had left him very little to answer. Sir Bradford Leslie had spoken about the liability of such caissons as were used at the Hawkesbury Bridge to "blow" when the level of the water inside the dredging-well was lower than that outside. The difference of the water-level, however, was of small extent, and there was no reason to believe that blows had occurred in this case. Speaking from memory, a main span of the bridge as launched weighed about 950 tons, and, adding the roadway girders of the overhang, which were fixed after launching, and the permanent way throughout, the weight as completed would be about 1,020 tons. It was difficult to conceive how both Mr. Ewing Matheson and Mr. Maynard got the notion that such a bridge as this, or indeed any important bridge, had been founded on mud, in the face of the plain statement, repeated twice in the Paper, that the caissons were sunk well into the sand; and this was especially mentioned with regard to pier No. 5 to which Mr. Maynard alluded. The Hawkesbury River opened into a wide estuary immediately below the bridge, so that floods had little or no effect there, not so much in fact as the tide, and there was practically no scour of the mud in its bed from either cause. Mr. Ewing Matheson had stated his impression that the American constructors had information that was not available in England, in regard to the exact nature of the soil. As far as the Author was aware they had no advantage over others in this respect. Mr. Burge.

Correspondence.

Mr. Bouscaren. Mr. G. Bouscaren observed that the Papers of Messrs. Burge and Walton, describing the methods of construction used for the deep foundations of the Hawkesbury and Dufferin bridges, were of special interest as dealing with an engineering problem, of which there were, as yet, only a few applications, and which required, perhaps to a greater extent than any other, the forethought in designing, and the fertility of expedients and perseverance in execution which characterize all great works of engineering, and often wrenched success from the very brink of disaster and failure. When considered in connection with the similar works done for the Poughkeepsie and Hooghly bridges, they were particularly instructive, as illustrating how the same results could be accomplished by different means, and also from the similarity of the accidents to the caisson in all these undertakings. Ample security in the strength of the anchorage and fastenings designed to hold and guide the caisson whilst sinking, until it was firmly engaged into the bed of the river, seemed to be the first desideratum. The difficulty experienced in guiding the caisson, and keeping it in a vertical position whilst sinking through the mud and sand, was suggestive of more effective means being employed to overcome

Mr. Bouscaren. the unequal friction by unequal loading or pressure at the four corners of a rectangle over the pier, in addition to the filling of the side pockets, or to decrease and equalize the friction by the use of water-jets through the outer skin, which did not appear to have been tried in either case. The use of water-jets also at the base of the dredging-chambers, would probably facilitate the dredging and the regularity of descent of the caisson, and might be of great assistance in removing boulders and other obstructions from under the cutting edge of the caisson. Such contingencies, which fortunately did not seem to have occurred in either of the four great undertakings where the method of dredging in open caissons were resorted to, were liable to happen, and would be very unpleasant to meet, where the obstructions would present themselves beyond the limit of working depths in compressed air. Tree trunks were found in the Mississippi river far beyond this limit; and he might mention, in this connection, his experiences with a huge cyprus trunk under an open cylinder, which was sunk for the foundation of the bridge across Pearl river in Louisiana, for the crossing of the New Orleans and North Eastern Railway. Although scarcely 30 feet under water, this tree caused a great deal of annoyance and delay. It lay in a horizontal position, with the ramification of its roots under about two-thirds of the circumference of the cutting edge of the cylinder. It was finally removed by cutting the roots and pulling them out singly, the water-jet being of great assistance in loosening the roots from the sand. He fully agreed with Mr. Burge in respect to the sizes in plan to be given to the caisson as compared with those of the piers, and thought that notwithstanding all the precautions which might be taken to prevent the shifting and canting of the caisson, the possibility of these accidents remained, and should be allowed for, by ample provision in the sizes of the caisson. The extra cost, incurred by the increase in sizes, might possibly be compensated by filling the greater part of the dredging chambers with sand instead of concrete, provided that the side pockets were filled with a good quality of concrete, and the bottom and the top of the dredging chambers were also sealed with concrete. The methods applied for the erection of the superstructure at Hawkesbury and for the Dufferin bridges were probably the best suited to each case, seeing that independent spans were to be used; but where cantilever spans are permissible, the method of building out with travellers, as applied at Poughkeepsie, was certainly the safest, if not the cheapest. The pontoon plan was open to objections for the risks arising from the currents and winds, and from the

Mr. Bouscaren. fact that, under certain circumstances, the breaking of a hawser, or a wrong movement at the critical moment, might cause the loss of an entire span.

Mr. Carey. Mr. E. G. CAREY stated that the steel in the superstructure of the Hawkesbury Bridge was similar in all respects to that employed in the Forth Bridge to resist tensile strains, and had an ultimate strength of 30 to 33 tons per square inch, with an elongation of at least 20 per cent. in a length of 8 inches. Strips with a minimum width of 1½ inch, and either cold or tempered by being heated to a bright red, cooled in air to a dull cherry red, and plunged into clean water at 82° Fahrenheit, being required to bend without fracture to a curve whose inner radius equalled 1½ time the thickness of the bar or plate. In practice, the elongation considerably exceeded 20 per cent., ranging as high as 30 per cent., whilst the material would frequently bend double without fracture. One tensile test was made from every fifty bars or slabs; whilst a cold or temper bend test was made from each bar or slab. The following analysis, selected at random, represented the average chemical composition of the material:—

	Per cent.
Carbon	0·200
Silicon	0·031
Sulphur	0·038
Phosphorus	0·047
Manganese	0·692
Copper	trace
Difference	Iron

The plates were rolled in double widths and up to 40 feet long, varying from ⅜ inch to 1¼ inch in thickness. The rivet-steel had a tensile strength of 26 to 30 tons per square inch with an elongation in 8 inches of 20 per cent., ranging in actual practice as high as 36 per cent. Bend tests similar to those for plates and bars, already quoted, were required. The above material was rolled by Messrs. David Colville and Sons, Dalzell Steel Works, Motherwell, from material manufactured by the Siemens-Martin open-hearth process.

Mr. Clarke. Mr. THOMAS CURTIS CLARKE believed that it might interest the Institution to know the position of "Constructing Engineers," in the United States, meaning by that title those engineers who both designed and executed their own works. During the rapid development of iron-bridge construction in the United States certain engineers, whose attention had been particularly directed to that branch of construction, soon found that a sufficient clientage could not be obtained if they confined themselves merely to making

Mr. Clarke. designs for others to execute. In order to gain the support of railroad managers, they found it necessary to guarantee the correctness of their figures, by offering to construct the work for those figures. This led to the necessity of their controlling both a large manufacturing and an erecting plant. Steel and iron could always be bought in the open market, but this control of the manufacture and the erection of bridges enabled them to name with certainty both the cost of a bridge and the time in which it could be built—an equally important consideration with railroad men in America. This developed their constructive skill and ability on the same lines, and they learned to make designs of any required strength at the least cost; and it was one of the reasons why the combined design and tender of the Union Bridge Company for the Hawkesbury Bridge was found acceptable. To show how close estimates could be made in advance, here were the estimated quantities of the Hawkesbury Bridge made in 1885, and the final measured quantities of 1889:—

	1885.	1889.
Tons of steel in grades	6,200	6,320
„ „ caisson	1,600	1,668
Cubic yards of concrete	27,600	26,593
Masonry	4,900	5,630
The excess of masonry was due to a modification of the plan.		
Time	30 months.	34 months.

Mr. Burge criticized the form of the caissons. No doubt they could be improved. Engineers learned from every piece of work executed how to do better next time. But after all, it must be allowed that the plan was not an unsuccessful one. The piers were sunk to depths never before reached, not enough out of position to do any harm, and in the estimated time, as the four months' excess was admitted to be due to unavoidable causes, and not to any fault of design or execution. The rapid sinking of these caissons was due to the fact that they were so designed as to carry a large mass of concrete between the inner and outer skins, forming part of the permanent construction, and by its weight overcoming the side friction, and preventing the necessity of the slow and expensive process of piling rails or other weight on the caissons to sink them. This excess of downward pressure had one disadvantage. Where the material passed through was softer on one side of the caisson than on the other, it naturally worked over to that side, and this was what actually took place at pier No. 5, possibly increased by the batter on the sides of the caisson, as pointed out by Mr. Burge. This, however, was the

Mr. Clarke. only pier which gave trouble, and the rapid execution of the work was due to the great weight of the caissons, exceeding the resistance of the side friction in the proportion of nearly 2 to 1. The deductions to be drawn by engineers from the sinking of the deep caissons at Hawkesbury were :—

1. Open dredging, rather than compressed air, should be used wherever the material was soft enough to dredge.

2. Caissons should be designed so that the downward pressure should far exceed what was believed to be the probable side friction, varying from 300 lbs. per square foot of rubbed surface in mud, to 500 to 600 lbs. in sand, and 700 to 800 lbs. in clay.

3. The sides of the caissons should be made vertical, and the bore should increase by offsetts rather than by batter.

The designers of this bridge felt some pride in their design, and as from the Paper of Mr. Burge no one could possibly tell who the Hawkesbury Bridge was designed by, Mr. Clarke might be permitted to state that the plans of the caissons were designed by Mr. Charles Macdonald, M. Am. Soc. C.E., and Mr. Thomas Curtis Clarke, M. Inst. C.E., conjointly, a year before the Hawkesbury competition, for a proposed deep foundation in the United States. Complete plans and models were made of this, and it was adopted for the Hawkesbury Bridge, and built without charge. It might also be of interest to know that this plan coincided almost exactly with a sketch made by Sir Benjamin Baker two or three years before, which, of course, was not seen until after the award. The erection of the superstructure on a large pontoon, was also designed by Messrs. Macdonald and Clarke. Small spans had been erected on a number of pontoons; but nothing like this, requiring the loaded pontoon to be carried so far, had been done before. Its success was due in a large measure to the foresight and ability of Messrs. Ryland and Moore who carried it out. The prompt action taken by them at span C, as described by Mr. Burge, saved a heavy loss. It seemed "curious" to Mr. Burge that though the tender was made by an American engineer, the whole of the steel and iron, except the eye-bars, were provided by the United Kingdom, where also it was manufactured. There was nothing curious or strange about this. Bridge shops in the United Kingdom were perfectly able to work to drawings, and the drawings were made by the Union Bridge Company's office. Where the bridge was manufactured, depended on the price of iron and steel at the time. Sir William Arrol, who constructed these girders at his shops in Glasgow, and constructed them very well, would have made the eye-bars also, if it had been

Mr. Clarke. worth his while to put in the expensive plant required for one order. In making the designs Mr. Macdonald and he carried out an idea, which at that time would not have met the approval of American engineers, namely, to rivet all connections except those uniting the main diagonals to the chords, where pins, as large as 7½ inches in diameter were used. It might be of interest to refer to the tests of the eye-bars, made with the 600 tons testing machine[1] at Athens, Pa., U.S. The steel bars were made by the Steel Company of Scotland, upon the following specifications of the Consulting Engineers, Sir John Fowler and Company. "Strips cut lengthwise or crosswise to have an ultimate tensile-strength of not less than 30 tons, and not exceeding 33 tons per square inch with an elongation of at least 20 per cent. in 8 inches. Tests of full-sized bars will be made as follows:—Out of every lot of one hundred and three bars, three shall be selected by the inspector. If two out of three break in the body of the bar with a stress of not less than 28 tons per square inch, nor a less elongation than 10 per cent. in the length of bar, then the lot of one hundred will be accepted if otherwise satisfactory. If but one out of three so breaks, the testing shall be continued at the expense of the manufacturers until the required proportion of two-thirds be reached. But if five bars break in the head without developing the above specified qualities, then the whole lot of one hundred shall be rejected. All pins shall be accurately turned to a gauge. Pin-holes shall be drilled to fit the pins with a play of not over $\frac{1}{50}$ inch. The length of eye-bars shall be adjusted to such accuracy that when ten bars are piled upon each other the pins shall pass through the holes at both ends. All eye-bars shall be annealed after manufacture."

Mr. Collingwood. Mr. F. Collingwood directed attention to the desirability in all caisson work of introducing a considerable amount of what he would name safety-bearing surfaces. For example, in the Brooklyn caisson of the East River Bridge, when the cutting edges were sunk 6 inches into the earth, they would furnish about 550 square feet of bearing. The five bearing frames, 2 feet thick, furnished about 1,000 square feet more at the same time. In the New York caisson, when the cutting edge was buried 6 inches, instead of 1,550 square feet, about 2,500 square feet came into bearing; 3 inches further penetration brought about 650 square feet more, and 2 feet penetration brought in all nearly 5,000 square feet. The Brooklyn caisson was sunk only

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[1] Transactions of the American Society of Civil Engineers, vol. xvi. 887, p. 1.

Mr. Collingwood.

44½ feet, and was at one time 12 inches out of level; while the New York caisson only touched mud at a depth of 37 feet, and was sunk through worse material to 78 feet depth, with but 9 inches variation from true level. Experience there showed the importance of this rapid increase in bearing surface, as it prevented downward movements from being so great as to cause wide deviations from verticality in case of variations in hardness of the material passed through. It was also a great safeguard against damage to the caisson, or serious derangement, in case of a blow-out. The reason given for the easterly movement of caisson No. 5 of the Hawkesbury Bridge might be correct; but the Brooklyn caisson, with a batter of 1½ inch per foot, moved away from, not towards, the harder material, a distance of 2 feet in a descent of 26 feet. This he attributed, however, to the pressure from a greater height of bank on the hard side. If it was true that the softer material filled in more quickly and closely against the caisson, might not the excess of pressure thus caused by it, rather than its resistance, have produced the movement noted. This would seem to be indicated by the decided success in correcting the positions of the caissons of the Dufferin Bridge, by external pressure. This latter experience would suggest also the use of the righting moment of weighted cantilevers on the high side of caissons, when out of the vertical. It was evidently not wise, in sinking very deep foundations, to limit too closely the size of the caissons. If the material to be passed through was at all yielding, the wedge-shaped cutting edge would readily penetrate from 1 foot to 2 feet into it before the inertia of the moving mass was overcome. Under such conditions, any inequality in hardness caused at once a difference in the amount of penetration at the sides or ends. This, though perhaps but a few inches, becomes feet in the amount of deviation caused at the top of a tall caisson, being more and more as the caisson was narrowed. A uniform progression in settlement was always to be preferred to extensive movements, and rapidly-increasing bearing surface tended to prevent the latter. The lateral, as well as longitudinal, distribution of wells recommended by Mr. Burge, was undoubtedly to be desired.

Mr. Cooper.

Mr. Theodore Cooper noticed that the Author of the Paper on the Hawkesbury Bridge said: "It is a curious circumstance connected with this bridge that, though the successful tender was made by an American firm, the whole of the steel and iron, except only that of the eye-bar heads, was provided by the United Kingdom, where also it was manufactured." The steel for the

Mr. Cooper. eye-bars was made in Scotland; the bars, however, were shipped to the United States for the purpose of forming the eyes according to the accepted practice of American bridge manufacturers, namely, upsetting and forging them from the solid bars. This work could not have been done satisfactorily in the United Kingdom, as no firm was, he believed, prepared to make eye-bars of the sizes required, which would comply with the ordinary requirements of American bridge practice, namely, that the bars should break preferably in the body of the original bar rather than at any point of the head or neck; and that the fulfilment of this requirement should be determined by testing to destruction a certain number of the full-sized bars. The girders and other riveted parts of this bridge were made in the United Kingdom; but it was understood that the cost of manufacturing such parts was fully up to the cost of similar work at the home works of the American firm. The only advantage, therefore, which the American firm obtained, over manufacturing the whole work in America, was the better market rates for structural material then existing in Great Britain, and possibly better facilities and rates for shipments of material to New South Wales. During the past year the market rates for structural material would have been more favourable to the United States than to Great Britain. It was curious and also very interesting to note that, with the advantage of the same markets, the tenders of firms in the United Kingdom for this work ranged from £50,000 to as high as 100 per cent. in excess of the tender of the successful American firm. The only advantage remaining to the American firm must have been its wider knowledge and experience in works of this magnitude, and its ability to minimize the risks involved in the execution of works of this character. The bridge engineer and bridge manufacturer were recognized specialists in America. They had built within the present generation upon the 160,000 miles of railroads in the United States over 3,000 miles of bridges. In addition, they had built a very large mileage of bridges upon the highways, much more difficult to estimate, but exceeding several times the mileage of railroad bridges. As this amount of experience comprised foundations and structures of all kinds and magnitudes, and as the greater portion of such work had been done under close competition, the American bridge-builder was prepared to estimate very closely the cost of any such piece of work as the Hawkesbury Bridge. The description of the Dufferin Bridge and its erection supplied an explanation why American bridge-builders could underbid, to the extent of 50 per cent., Mr. Cooper.

their competitors in the United Kingdom, on works of magnitude involving great risks. A method of erecting a span 355½ feet long, involving incessant work throughout the twenty-four hours of each day, for one hundred and thirty-two days, over a river having the characteristics of the Ganges, would certainly justify an increased percentage of 20 to 100 over the methods employed by American engineers. The erection of bridges over the Ohio, Missouri, Mississippi, and other great rivers of America was subject to similar risks as were to be expected over the Ganges and other rivers of India. The erection of the two channel spans of the bridge over the River Ohio, near its mouth at Cairo, Illinois, was a fair example of the possibilities of the American system of construction and erection. These spans were each 518 feet 6 inches from centre to centre of the end pins, 61 feet deep from centre to centre, 25 feet wide from centre to centre of the trusses, the panel length 30 feet 5¾ inches, and the total weight of one span, 2,055,200 lbs. The first span was erected in six days. Then the false works were taken down, the supporting piles drawn and re-driven for the second span, the false works again put up, and the second span erected. The whole time, covering the erection of the two spans and moving the false works, was one month and three days, including five days' lost time, waiting for the completion of the masonry. The false works and traveller were in position, ready to commence the erection of the second span, by 2.30 P.M. on the 30th of October, 1888. At 2.50 P.M. on the 3rd of November, the trusses of this span and the top tracing were all connected. No work was done at night. The material was run on trucks about 1,025 feet, from the storage yard to the nearest end of the span being erected. About twenty-four men were employed in delivering the material, and fifty in erecting the parts and connecting them together. The false works stood about 104 feet above low water. The bents being 72 feet high above the capping of the piles, the depth of water at its low stage was about 20 feet. The piles were from 50 to 75 feet long. When a thoughtful consideration of the relative cost and hazard of these two examples was taken, it did not appear so curious or unintelligible why an American firm of the ability and experience of the Union Bridge Company should be the successful competitor, on a work it was so well prepared to estimate and execute. Nor need it be matter for future surprise should American engineers and bridge-builders be able to meet successfully their competitors of the United Kingdom whenever the opportunity was given for a fair and free competition.

Professor Gaudard. Professor J. GAUDARD could not refrain from expressing his admiration of the foundation works of the Hawkesbury and the Dufferin bridges, which marked a great advance upon all examples cited hitherto, as of exceptional magnitude in the depth of water or of soil. Summarizing several of these cases, and noting the great mortality among the workmen, attendant upon the employment of compressed air in foundations below 100 feet in depth, he referred to the boring-head and hydraulic extraction adopted at the Gorai Bridge, and to the plan adopted by Mr. Jandin at the bridge at Palma del Rio over the Guadalquivir, by which work might be carried on at a depth of 130 to 160 feet in permeable soil, and to the occasional execution of works in the open air with provision for the use of compressed air, if necessary. He thought, however, that provision for manual labour was required only to a certain depth where descent was still in progress, and where it was necessary to examine the soil below the pier; and that this limit was certainly not in excess of 100 feet. Below that point the conditions of stability did not appear to make such examination requisite. The resistance of the soil below the pier, the loss of weight by immersion, and the lateral friction, the three elements in the case, were all capable of exact expression and graphic demonstration. Thus it could be determined in a general way what would be the requisite depth of the foundations for the resistance of the soil, and the lateral friction to support the pier in a state of repose, and without adding to the normal pressure upon the site of the foundation, in accordance with the formula given by Rankine. In dealing with the pressure of the soil, as affecting all parts of the foundation, it was, of course, assumed that the ground must be treated as sufficiently fluid for the pressure to be so exercised; but in dry soil the deficiency of lateral pressure would be more than compensated by the resistance to direct compression and to disaggregation. He agreed therefore with Mr. Walton, that, in the case of the Dufferin Bridge, the friction and hydrostatic displacement would probably not exceed one-half the total resistance, leaving the base to support the other half of load. The principal risk, in sinking high and relatively slight piles or piers, was the lateral displacement or deviation from the perpendicular, in an uneven soil. This might be caused by the current, but in such case it would affect the operation in its earliest stage when the pier was simply being guided into place, and its position could be rectified. There was also the yielding of soft beds to the scour of the current, and obstacles and sources of dislocation in varying strata, or the inclined surface of a hard bed.

Professor Gaudard.

Mr. Walton had referred to the success of the means of replacement employed in the Dufferin Bridge, and other methods might be equally successful in other cases. The difficulties occasioned by slipping of the soil at the base of the pier, in beds of heterogeneous formation, were very clearly shown in Mr. Burge's Paper; and impressed Professor Gaudard with the importance of avoiding the bottom outward splay; as confirmed also by the Saint Leger Viaduct on the Grande Ceinture Railway, of Paris, which afforded an instance of the risks of a pier sinking too rapidly through a soft and yielding soil. A pier with vertical sides had considerably the best chance of retaining the perpendicular in its descent, provided it were well guided for the first 30 feet or so. It was to be regretted that in the greater depths of water, guiding piles should be abandoned; for where the depth precluded the use of timber, iron piles could be employed, and properly secured. It might perhaps be objected that such piles would have shared the fate of the cribwork in the Hawkesbury Bridge, but they would be planted more deeply and firmly. He thought it would be eventually feasible, by the adoption of a temporary outer cylindrical casing on the Gaertner system,[1] to diminish the lateral friction during the sinking of the pier, while restoring its full effect on the stability of the structure when the operation was completed. The method of sinking the foundations for the Blackfriars Bridge, presented itself to him as open to some criticism, but was doubtless necessitated by the circumstances of the case. The question of securing the ironwork to the masonry appeared to have been successfully dealt with by Mr. Cruttwell; but it was a most difficult one, and disastrous consequences might accrue in case of insufficient provision for alterations of temperature, as happened to several bridges, where abutments had been split throughout their height. He could not understand, in the case of bridges such as those over the Hawkesbury and the Ganges, why independent girders were employed, since the system of cantilever bridges had been so successfully carried out. The exactitude of the measurement of the width of the Ganges confirmed the excellence of the method adopted by the verification of the calculation.

Mr. Gillott.

Mr. T. Gillott remarked that the amount given as the tender for the Hawkesbury Bridge, namely £327,000, appeared to be a very moderate sum; but he should be glad to know the actual cost, and also the weights and quantities of the respective parts, as in these

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[1] Wochenschrift des österreichischen Ingenieur- und Architecten Vereins, 1885,

Mr. Gillott. items the Paper on that bridge was deficient. The account of the erection of the Hawkesbury Bridge was most interesting; great skill had been exercised, and the defective arrangements of the caissons with respect to the outward splay, and the position and number of the dredging tubes (p. 7) which were candidly pointed out, would serve as valuable hints to designers of future works of a similar character. From the comparative ease with which Nos. 1, 2 and 3 caissons were founded, the small range of the tide (7 feet), and the velocity of the stream (3 to 4 knots per hour), he should be glad if the Author would state whether, in his opinion, there was any reason for the adoption of such large spans as 415 feet, and whether the stream could not have been bridged at a less cost by shorter spans. The settlement of the girders for the Hawkesbury Bridge, as apparent from the built camber of 12 inches being reduced to 3½ inches when the spans were left on the piers, seemed to him more than should have been expected, and appeared much more in proportion than that of the Dufferin Bridge, which, being built with a camber of 9 inches, had still 4½ inches left when under the total dead-load. Owing to the great depth of the pin-connected girders of the Hawkesbury Bridge, he should have expected the respective settlements of that and of the Dufferin Bridge would have been about the same, and should be glad if the Author of the first Paper would state the deflections of the Hawkesbury Bridge under a rolling load, to compare with the deflection of 1·45 inch, given for the Dufferin Bridge, as Mr. Gillott believed that the net strains were the same in both bridges. The use of power riveters during erection would no doubt tend to place riveted structures at greater advantage compared with pin-connected girders; and the greater stiffness of the former could now be secured without involving undue expense for hand-riveting on the site.

r. Parkinson. Mr. H. H. Parkinson stated that, in sinking brick wells in India he had found that the porous nature of the brickwork caused the well, when emptied of water, to suck so large a quantity of moisture from the surrounding soil, as to very much increase the friction while sinking; and that, in the beds of some rivers, a ring or cylinder of partially dried soil immediately surrounding the well clung to it with such tenacity as to follow it down in its descent, breaking away from the surrounding wetter soil in preference to parting from the sides of the well. In the event of the corner of a large caisson encountering a rocky projection, during its descent, he should sink a well or second caisson inside the first, at the opposite end to where the impediment occurred in the first; and, when a sufficient depth had been attained, join the two together

Mr. Parkinso by a step, or bring the upper caisson to bear on the lower by means of cross beams. When seeking for water in India at any considerable depth, he had sunk wells in two parts, the first well of large diameter, intended to go down to the water-bearing strata, by which time it would be pretty well knocked about, and might even have lost its shape, having numerous horizontal partings in the brick rings, and possibly having become out of plumb. He then found it convenient to start a fresh well at the bottom of the first, correcting any crookedness, and obtaining at the same time, a platform at an intermediate level for the men to work from.

Mr. Burge. Mr. C. O. Burge, in reply to the correspondence, said he was pleased to find that the only fault found with his Paper was that it was too short. In describing a structure of such magnitude as the Hawkesbury Bridge, a Paper might be read on each of the parts into which the work would naturally be divided, namely: 1. The triangulations for determining the position of the piers—an operation of considerable difficulty; 2. The foundations; 3. The masonry; 4. The superstructure; 5. The method of launching the latter. Nos. 2 and 5, being of a specially novel character, had been principally dwelt on, the others presenting no very remarkable features differing from similar works already described. The want of information alluded to by Mr. Gillott and others had reference chiefly to the superstructure, which had been thus purposely left undescribed except in general terms. He was not aware of the actual cost of the work to the contractors. To the Government the contract sum of £327,000 had been increased by extras, ordered by the Government engineers, before the bridge was begun, to £340,000, and there was a slight increase to this on the completion of the work. With regard to the fixture of the spans at 416 feet: as the design formed part of the tender, it was a matter on which the contractors had to decide; but he believed that, owing to the difficulties which were to be expected in such unusually deep foundations, longer spans rather than shorter might have been anticipated in the successful design. With regard to the comparison between the difference of camber when built on the staging, and on taking their own weight, of the Hawkesbury and the Benares girders, Mr. Burge would remark that though the ratio of depth to span was greater in the former, the total span of the Hawkesbury girder was more than one-sixth greater than in the Indian work; moreover, the former was a pin-connected truss, each eye in the top and bottom boom being made $\frac{1}{64}$ inch larger than the pin passed through it; and to drive the pin easily a substantial erection camber had to be provided.

Mr. Burge. According to the description, the Dufferin Bridge was a riveted structure. The deflection of the Hawkesbury truss under a rolling load of three Consolidation engines, followed by a heavy train on each line of rails, was 2½ inches, or $\frac{1}{1,964}$ of the span. The 9 tons per square foot, given as the maximum pressure on the foundations, referred only to the heaviest pier, and from this would have to be deducted the buoyancy and the skin friction.

11 March, 1890.

SIR JOHN COODE, K.C.M.G., President,
in the Chair.

The discussion upon the Papers by Mr. C. O. Burge, Mr. F. T. G. Walton, and Mr. G. E. W. Cruttwell, on "Railway Bridges," occupied the whole evening.

18 March, 1890.

SIR JOHN COODE, K.C.M.G., President,
in the Chair.

(*Paper No. 2451.*)

"Lough Erne Drainage."

By JAMES PRICE, Jun., M. Inst. C.E.

THE subject of Arterial Drainage has become one of great interest in Ireland of late years, first on account of the increased floods in the main rivers, owing to the more rapid discharge of water off the uplands from improved drainage; and, secondly, from the increase of flood-water, owing to the reduction of absorption, resulting from the diminution of timber since the year 1847. In the Erne districts there were special causes of increased flooding, first, the extensive arterial drainage works carried out at Lough Oughter (which discharges into the Erne); and secondly, the obstruction of the water at the crest of the natural controlling weir of the lake at Belleek, by the barrier erected to increase the water-power for the pottery works in 1852 and 1853, and by the improvements to the rampart walls of an old eel-weir situated on the crest of the same falls.

The Erne drainage-basin extends in a north-westerly direction for about 90 miles, with a maximum width of 25 miles, containing 1,056,160 statute acres, of which about 1,000,000 acres are land. The "lip" of this land is 154 to 155 feet above Ordnance datum, and the crest of the watershed is 300 feet, except where the Donegal and Sligo hills rise to 600 feet and 1,200 feet respectively; but the smaller portion of the drainage area lies between these higher levels and the lakes. The Upper and Lower Loughs Erne, connected together by two winding channels, each about 11 miles long, formed two natural ponds, into which the drainage of the district emptied itself; the only outlet being over the crest of the rock barrier at Belleek, about 3½ miles from the north-western extremity of the Lower Lough Erne (Plate 6, Fig. 1). In extremely dry summers the two lakes stood practically at the same level of 151 feet, the barrier at Belleek being about 150 feet above datum, the dry weather yield of the land being sufficient to keep up a head of about 1 foot over the limited width of the crest then available for discharge. In ordinary summers, with moderate rainfall, the lower lake stood at about 152 feet, and the upper lake at 152·50 feet. During autumn and winter floods, the lower lake rose from 156 to 157 feet, and the upper lake from 158 to 161 feet, submerging about 17,000 acres of meadow-land, and choking the drains of some 15,000 acres of land suitable for tillage. The Erne drainage basin consists almost entirely of calp limestone (except at the north-west where a little gneiss and granite crop up); the rock is fairly absorbent, being a mixture of solid limestone with shaley beds. A glacier deposit of blue boulder-clay overlies the limestone; and a calcareous alluvial deposit lies on this clay in the upper lake district, with occasional layers of peat.

Works combining the drainage and navigation of Lough Erne were first proposed in 1840, when Mr. Rhodes (Engineer of the Shannon works) made a plan. In 1847, Mr. Forsyth (Engineer of the Board of Works) proposed a scheme, with a lock and weir between the lakes; and in 1852, Mr. Roderick Gray, County Surveyor, made out a plan on the same lines. In 1867, Mr. James Lynam, Engineer for Mr. J. Gray Vesey Porter, of Belleisle, produced a plan without a lock between the lakes, which was reported on in 1870 by Mr. S. U. Roberts, M. Inst. C.E., Inspecting Engineer of Drainage for the Board of Works, who agreed with Mr. Forsyth "that to maintain a uniform level at all times of the year, from Crom Castle, at the head of the Upper Lake, to Belleek, without either ruining the navigation in summer, or submerging the low lands in winter, is wholly impracticable within reasonable outlay." Mr. Lynam eventually produced a scheme having an intermediate lock and sluices at Belleek; and Mr. James Barton, M. Inst. C.E., also prepared a somewhat similar, but superior arrangement. Both these schemes contemplated reducing the level of the lower lake 2 feet, to form a reservoir for floods, which the proprietors along the lower lake considered would be injurious to their navigation, and to the appearance of their demesnes along the lake. Both schemes, however, though approved by the Inspecting Engineer of the Board of Works, were abandoned.

The conclusions arrived at from the various inquiries were:— (1) That, for the Erne basin, the usual "Fen district" flood of ¾ inch of rainfall per day would be much too high, and that the maximum flood might be safely taken at 0·25 inch of rainfall per day, or 750,000 cubic feet per minute. (2) That no scheme which spoiled the navigation would receive the assent of the residents on the lake shore; and that it would be considered objectionable to have any intermediate weir between the lakes, or to reduce the level of the lower lake below 152 feet above Ordnance datum, or the upper lake below 152·50 feet. (3) That if the level of the upper lake were not allowed to exceed 155·50 feet, about 14,000 acres of land would be saved from flooding; and that if the lower lake did not exceed 153 feet, 3,000 acres more land would be secured. The relief from periodical flooding would save the hay crop every fifth year, worth at least £3 per acre, amounting to an annual saving of 12*s.*, for which the tenant might reasonably be taxed at the rate of 6*s.* per acre on the land preserved from flooding. The minimum summer flow was taken at between 80,000 and 100,000 cubic feet per minute; but no actual measurements of low-water flows had then been made on the Erne.

In December 1878, Mr. J. Gray Vesey Porter requested Mr. James Price, M. Inst. C.E., to prepare a scheme combining the drainage and navigation of Lough Erne, without an intermediate weir. The problem was to maintain an unbroken sheet of 50 miles of water, so that the upper portion should not fall below 153 feet, nor rise above 155·50 feet over Ordnance datum, and that the lower portion should range between 152 and 153 feet; the flood-discharge required being 750,000 cubic feet per minute, and the dry-weather yield 80,000 cubic feet per minute. With one weir, either at Belleek, or at the exit from the lower lake, 3½ miles above Belleek, this would have been impracticable, as the water on such a weir could not exceed the level of 153 feet without commencing to flood the lower lake lands; and in order to maintain the level of the water on the upper lake at 152 feet (1 foot under the proper navigation level), the crest of the weir must not be lower than 151·75 feet. The head for discharging floods would then only be 1·25 foot, necessitating a weir 2,500 feet wide. Such an erection at the extremity of the lower lake would also have implied the loss of 3½ miles of navigation. It would also have been ruinous to the milling and fishery interests down stream, as a north-west wind would blow back the lake in dry weather so as to leave the crest of the weir quite bare, and thus tend to drain the river for 7 miles below it; for a strong wind against the stream affects the level of the lower extremity of the lake from 6 to 12 inches. The placing of such a weir at Belleek would have been still more difficult, as, besides the want of width, the only head (2·50 feet) available would have been distributed among the channels between the lakes and the channel above Belleek, requiring a large increase of section throughout.

The only solution of the problem seemed to be the adoption of some removable weir at Belleek, creating when opened an artificial fall from the lower lake to that point, and improving the channels between the lakes. Mr. Price, after examining the different forms of sluices in Holland, France, and the United Kingdom, and the dams on the Seine, came to the conclusion that large sluices with free rollers, as designed by Mr. F. G. M. Stoney, M. Inst. C.E., were the most suitable. He also decided that the lower lake should be kept as nearly as possible, at all seasons, at 152·66 feet above Ordnance datum, or about 8 inches above the average summer water-level of previous years. A fall of 4 inches was allowed between the lakes in summer, making the level of the upper lake 153 feet. It was assumed, from previous evidence, that the flood from the upper lake would not exceed 600,000 cubic feet per minute; and the flood from the lower lake district was taken at 150,000 cubic feet per minute; but as the flood from the lower district would run off much more rapidly than that from the upper, owing to the greater fall in the lower tributaries, this flood would be discharged before the upper lake flood arrived, and therefore 600,000 cubic feet per minute was taken as the maximum flood of the district. The sluices, moreover, combined with the large pondage area of the lower lake (27,643 acres), and the upper lake pond of 9,453 acres, gave great power of anticipating a flood, and drawing some 6 inches from the lake-surface during the time intervening between the commencement of heavy rain, or the first telegram to Belleek of a rise on the upper lake gauge, and the actual arrival of the flood-water from the upper to the lower lake. The section adopted for the channel from the lower lake was 250 feet by 11 feet, a fall of 1 foot per mile being allowed. On the channels between the upper and lower lakes, consisting of numerous pools separated by fords, different falls and cross sections were adopted, according to the forms of the natural sections and of the surrounding lands, such as 200 feet by 10 feet, and 150 feet by 12 feet, with falls up to 2 feet per mile (Plate 6, Fig. 1). The total fall allowed between the lakes in maximum flood was 2 feet 6 inches. The outlet channel from the lower lake, for about 1,000 feet above the sluices, was gradually reduced in section and increased in fall, until the water was to have been conducted to and discharged through three sluices, 29 feet wide, running 8 feet deep; the short channel below the sluices was further narrowed to the old bridge of Belleek, where the stream passes at high velocity through a natural rock-cut channel 30 feet wide and 28 feet deep, from which point to the sea the river has a free fall of 140 feet in 4 miles.

This was the scheme on which an inquiry was held at Enniskillen early in 1880 by Mr. Hawksley, Past President Inst. C.E., on behalf of the Irish Board of Public Works; and the scheme was passed, with an order from Mr. Hawksley, that £10,000 additional should be expended in putting up one more sluice at Belleek, and increasing the sectional areas of the channels in some places.[1]

The flooded lands were about 13,468 acres, and the soaked lands 3,798 acres, making a total of 17,266 acres; and the improvement was estimated at £5,266 per annum, or 6*s.* 1½*d.* per acre. But the results of the drainage show that this sum was much too low. Moreover, it is only a valuation for increase of rent; but the lake-side farmers gain at least as much clear extra profit on their crops as the increase in annual rent. The adjacent districts, moreover, benefit largely in various ways from the drainage works, without contributing.

The boulder-clay of Lough Erne, through which at least nine-tenths of the excavations had to be made, is a stiff, hard, blue clay, nearly dry in the centre, having a specific gravity of 2·21. The boulders were generally limestone, and were raised by the dredger up to 3 tons; but larger ones were frequently met with, requiring blasting.

Messrs. Amos and Smith, of Hull, delivered on the lake the whole dredging plant for £14,000, namely, dredger capable of excavating 2 tons per minute (£6,500); two hopper barges (£4,000); steam-tug (£1,400); and spare gear, buckets, and stores (£2,100). Details of this plant are given in Appendix I. The dredger and barges were built on Lough Erne; and the tug was built at Hull, and brought, *viâ* Belfast, to the lake through the Ulster Canal.

The works were commenced in May, 1882; but the contractor, after three months, abandoned them, leaving his plant as a forfeit. The Drainage Board then decided to carry out the cutting of Belleek falls, and erecting the sluices, by day labour; and in September, 1882, the Author took charge as Resident Engineer. A channel, 22 feet by 6 feet, was first cut at Belleek, as ordered by the Irish Fishery Commissioners, to allow the ascent and descent of salmon independently of the sluices. This channel was opened early in November, to assist in letting off the flood, which had then risen in the lower lake to 156·66 feet above datum. The main cutting on the falls was then commenced on Belleek Island; and the work was proceeded with during the winter of 1882–3; but, owing to the almost incessant floods, little substantial progress was made until the end of February. By May, the whole was properly enclosed by dams; and the excavations were pushed on. In June, 1883, the erection of the sluices, which had been constructed by Messrs. Emmerson and Murgatroyd, of Stockport, and conveyed in parts to Belleek, was commenced under the superintendence of Mr. Stoney, and the sluices were opened in September. During the following winter and spring, the other works at Belleek were completed; namely, an accommodation cutting to supply the Belleek Pottery Works with water at the reduced level; the completion and erection of the steps in the salmon pass; and the rock cutting below the sluices, which latter was accomplished by opening the sluices at intervals through the day, then shutting them down, and putting men to work at the rock until the water again rose to the top of the sluices.

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[1] The Treasury arranged to lend the money, to be paid off, principal and interest, by an annual payment of 5 per cent. (since reduced to 4 per cent.) for thirty-five years. A special Act of Parliament was also passed for subsidizing the work with £30,000, on condition that the navigation was not only preserved, but improved. This £30,000 was to be provided by a grant from the Treasury of £15,000, and by a loan to be paid off by a small rate levied on eleven baronies adjoining the lake and interested in the navigation. A Drainage Board was constituted from amongst the landowners with Colonel Irvine as chairman.

Dredging the channels above Belleek was commenced in the spring of 1883, by swinging the machine on the fore anchor-chain, and travelling from side to side on the port and starboard anchors, taking up a bench of about 10 inches deep, and then dropping back and taking another 10-inch bench. After two months' trial, during which the output seldom exceeded 300 tons daily, and averaged only 200 tons, this method was abandoned. The mode subsequently adopted consisted in dropping the ladder to the full depth of the cutting to be made, and keeping it there, travelling the machine from side to side, cutting up the full height of the face at once. As the cutting advanced, the height of face gradually increased up to 5, 7, or 8 feet. On this system the output at once rose to 400 tons per day at a 5-foot face, and to 700 or 800 tons when the height of the face increased to 7 feet; in the softer clay or gravel, the daily output was from 800 to 1,200 tons. The top surfaces of nearly all the shoals to be lowered were paved with boulders, from several tons in weight down to a few lbs., tightly jammed together, the softer material having been washed out of the boulder-clay, and leaving this natural irregular pavement. When working in 10-inch benches, every stone embedded in the clay stops or turns aside the bucket, while, in the 5-foot face, only such stones as lie at the bottom are an obstruction; when once these are displaced, those at a higher level are undermined, and fall out of the face into the bucket. Again, at a 10-inch face, boulders displaced are pushed forward over the face, and form a wall of stones in front of the cutting, which is liable to injure the dredger's bottom when working in shallow water; while, at the 5-foot face, the boulders are caught against the face, and are lifted in the buckets. In this way boulders up to 36 cubic feet were raised in the buckets; and the lifting of a boulder of 8 or 9 cubic feet was a common daily occurrence.

The running of the engine is much more regular when working on a high face, as each bucket enters its work before the previous one has got free; and the "chucking" caused by each bucket biting in succession, as in a 10-inch face, is avoided. Also the strains are carried more directly to the driving tumbler; as at a low face the bucket does most of its work when behind the bottom tumbler, and the whole pull of the bucket-chain must pass round this tumbler, causing great friction on the shaft and chain pins. At the high face, the work is chiefly done as the bucket travels upwards, when it is nearer the direct pull of the chain. At a low face, as the reaction is horizontal, it exerts a strain on the main bow anchor-chain; whereas at a high face, most of the reaction is vertical, and is taken by the flotation of the hull.

After the completion of the works at Belleek in the spring of 1884, the men were sent to cut such parts of the channels leading from the lower lake to Belleek as could be excavated in the dry. Clay was taken from the shore of the river in shallow water, to enclose about 50 feet square of the area to be cut; and, from this, clay was taken to enclose 120 feet by 200 feet, thus approaching the centre of the stream. From here the main longitudinal clay dam was run, so as to leave two-thirds of the river for the discharge of water, one-third being ponded for work. The banks were made continuous down the stream, as when raised against the current, much of the freshly-deposited bank was washed away. Lengths of the river, ½ mile long, were thus enclosed. In depths not exceeding 4 or 5 feet little difficulty was found in thus making dams; but at greater depths, other methods had to be resorted to. In a 4-foot dam the clay stood at a slope of 1¼ to 1; but in greater depths the slopes flattened, the bottom 6 feet of the 10-foot dam taking slopes of 3 to 1, while the top 4 feet remained at 1¼ to 1. To prevent this spreading, stakes were driven; and for the deeper dams, the base was deposited from barges, and allowed to settle before the top was laid on. The closing of a dam in deep water was avoided, as the two soft toes of the slopes, charged with water, spread each other over the bottom of the river; and, even when a junction was made, it always formed a weak point, liable to slip.

Where an embankment must be made in water exceeding 8 feet in depth, the Author considers that the work should be commenced at the deepest portion, and extended in both directions into shallower water. The freshly excavated river clay was very good for dams; but when once soaked with water, it became too soft to use a second time, except for topping dams in summer, having plenty of time to dry before the rise of the winter floods.

The smaller enclosures were pumped by hand scoops, or hand chain-pumps; but all the main pumping was done by centrifugal pumps, driven by portable engines. Rubber belting was used, as it does not slip when wet, lasts longer, and costs only about one-third the price of leather belting. Details about the excavations are given in Appendixes II and III.

The main principle of the sluice-gates[1] is the reduction of friction by putting a train of 8-inch live rollers between the moving surfaces, kept in position by a chain passing under a pulley in the top of the roller-frame, one end being fastened to the fixed staging over the sluices, and the other end to the top of the sluice-gate, thus giving the roller-frame a velocity of half that of the sluice-gate. The lifting is done by a 4-inch double threaded screw of 2 inches pitch, at each side of the sluice, working in a nut on the sluice-gate; a ring of coned rollers is arranged under the screw-head on which the load is carried. Each alternate coned roller is turned smaller: the weight is thus taken on the larger rollers, and the smaller keep the distance, rolling in a reverse direction, thus obviating all sliding between the rollers. The gearing is arranged so that each sluice may be lifted by one man, taking about forty-five minutes to open fully one gate 9 feet. A small turbine is also provided, by which any one of the sluice-gates, or all of them, may be lifted simultaneously, at the rate of one sluice-gate in twenty minutes, or all in half an hour; but such speed is seldom used, as the sudden flood thereby caused in the lower river is dangerous to anglers, and otherwise inconvenient. Each of the four sluices is 29 feet 2 inches span, by 14 feet 6 inches high, capable of lifting to give a 9-foot opening. The sluices have been in use for six years, and have given the fullest satisfaction. The lifting is easy work for one man; and the motion is almost as easy under the full water-pressure as when the sluice is clear of water; indeed, the condition of the gearing affects the labour more than the water-pressure. In closing, the gates run down of their own weight when started. To prevent overwinding by the turbine, an automatic clutch is provided, which the gate disconnects when full open or shut. Each gate weighs about 13 tons; and no counterbalance is provided, which would, if used, have effected a saving in the gearing, and would admit of quicker motion. The cost of the four gates erected was £4,000, including royalty, but exclusive of masonry. The staunching at each vertical edge is effected by a 2-inch turned bar, which hangs freely from the gate in a V groove, one side of which is formed by a planed casting fixed to the pier, and the other by a similar casting on the gate, coming within ½ inch of each other at the angle of the V. The masonry piers are 45 feet by 10 feet 4 inches; they are boat-form in plan, and are faced with limestone ashlar, 18 inches thick, set in Portland cement mortar, and hearted with 6 to 1 concrete, with large stones interspersed. The piers are built thus up to 16 feet above the sills, on which are erected towers, 8 feet by 9 feet, to carry the bridging for the lifting-gear. The whole is let into the rock 2 feet below the sills.

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[1] *Engineering*, Sept. 19th, 1884, vol. xxxviii. pp. 268–270 and 272. *The Engineer*, Sept. 19th and Oct. 10th, 1884, vol. lviii. pp. 218 and 282–284.

The works in the upper lake, and between the lakes (which constituted about one-third of the whole), were carried out by contract by Mr. John Best, of Leith, commencing in 1885 (Plate 6, Fig. 1). The works were similar to those carried out by the Author; but in many of the cuttings the material turned out softer than was expected; and as there were several small cuttings, a good deal was shifted by barrows. The total quantity of material excavated in the drainage works was 915,568 cubic yards of clay, and 291,585 cubic yards of rock. The cost of these works much exceeded the original estimate, owing to large and unexpected claims for fishery rights, and to delays at the commencement, connected with the Act of Parliament, over which the Drainage Board had no control. The interference with these fishery rights consisted in the removal of two eel-weirs, for which large compensation had to be paid, and the alleged anticipated injury to the salmon fishery of the Ballyshannon Fishery Company, for which a large sum was awarded. Though, however, the rod fishery has been inconvenienced by the dredging, the take of salmon in the nets and boxes, which are the chief modes of capture, has increased since the work commenced, and is steadily improving. Drainage operations, indeed, generally improve salmon fisheries, by removing shoals at the mouths of the tributaries, where previously fish were unable to pass with safety to their spawning-grounds. In the case of Lough Erne, the salmon are now seen in numbers at places where they were rarely found formerly, and they also come up much earlier. The salmon fishery also became many times more valuable in the Lough Corrib district after that drainage had been carried out.

The works have cost £180,000, being £30,000 for navigation, and £150,000 for drainage; which latter comes to £8 12*s.* 6*d.* per acre reclaimed or improved. No credit, however, has been taken for the value of the dredging-plant which has been maintained in excellent order.

For several years two rain-gauges have been kept as the standards of the Erne district, one at Florence-Court, on the high ground south of Enniskillen, the other at Colebrook, about 8 miles north-east of Enniskillen; and the average of these gauges has been taken as the average rainfall of the district. A registry has also been kept for some years, at Belleisle, of the level of the upper lake on each day; also since the commencement of the works, a daily register has been kept at Roscor of the level of the lower lake. Since the beginning of 1884, a record has been kept of the height of each sluice as raised on any day, with the level of the water at the top of the channel leading to the sluices.

The discharge of the river was observed, in a fairly straight length of uniform section, by the following methods. (1) For small velocities, a disk of sheet zinc, 1 square foot in area, just floating, was attached by three strings to a line going round a pulley, which could be adjusted at fixed distances on a pole; and this line was attached to a spring-balance at the top of the pole. Observations were made in a boat, by fixing the pulley at any desired distance from the bottom of the pole, pressing the pole down to the bottom of the river, and noting the pull of the disk on the spring-balance. The velocity was calculated from the vertical head of water required to produce the pull observed. Many observations were thus made at different levels, and at different distances across the stream, which were plotted in curves; and the volumes taken out gave, in cubic feet per minute, the discharge at any point. For higher velocities, the rolling motion of the water caused such variable pressures that it was difficult to take a fair average reading of the spring-balance. (2) A bottle was weighted so that a fine string and small piece of cork kept it at a fixed distance from the surface, and the velocities were taken at different levels—a slow process, but fairly correct. (3) The bottle, as above, was so adjusted on the string as to float four-fifths of the depth from the bottom, the velocity being taken as the mean of the position. Observations were made at measured distances across the channel, and the results were plotted on the plan as a curve, the area of which, multiplied by the depth, gave the discharge per minute. (4) For comparative observations, the surface-velocity was taken, and the mean found from it by deducting 10 per cent. in deep channels of 10 feet, and 20 per cent. in shallow channels of 4 or 5 feet. No attempt was made to record the velocities by a current-meter, as it was considered that the great variations of "slip" of the screw, at the different velocities in the river, would lead to erroneous results. Nor was the method of loaded rods adopted, as such rods, if floated close enough to the bottom to give true results, would be liable to obstruction from stones or weeds. By a combination of the third and fourth methods, observations were made to ascertain the coefficient of discharge from the sluices, the third method being used as long as it was safe to work a boat near the sluices, and the fourth, with floats thrown from the banks, when the velocity became dangerous. Thus, with the sluice 8 inches open, the coefficient was 0·60, and when 8 feet open, 0·68, the variations being gradual between the two extremes. The width for the sluice-opening was 29 feet; and the head taken was the height from the centre of the orifice to the surface, when the sluice was discharging freely; but when back-water checked the discharge, the difference between the level of the water above and that below the sluice was considered as the head, the levels being always in a position where the surfaces were not affected by the motion of the water close to the sluices.

The Author has put in a diagram the results of the observations of rainfall, lake-levels, and sluice-discharges; the horizontal scale marking the periods, and the vertical scale the mean daily discharge, indicated by an irregular dotted line (Plate 6, Fig. 2). The full black line in the diagram represents, on the same vertical scale, the actual daily quantity of water coming into the lakes in the years 1885–88. This line of net supply is plotted above or below the line of mean discharge, according to the actual volume gained or lost in the lakes by an observed rise or fall of their levels, and is quite independent of the working of the sluices or of the pondage areas of the lakes, for it represents what would have been the accumulated supply from all the tributaries of the Erne. A line, connecting the lowest points of the thick black line, would represent the daily spring supply, or such as is not the result of recent rainfall; and the difference between these lines would represent the extra flow caused by rain. The line shown on the diagram in dot and dash represents the daily rain supply of the district, on the same vertical scale, as it would have flowed into the lakes with no loss from percolation, evaporation, or vegetation. The difference between the areas bounded by the lines of net supply and of rain supply represents the water lost by evaporation, vegetation, and percolation. If a, b, and c are the ordinates of each principal line, respectively, in the diagram, then (Plate 6, Fig. 2) :—

$$pq = a, \qquad pr = b, \qquad ps = c.$$

a = spring-water, in cubic feet per minute ;
$b - a$ = storm-water, „ „ ;
$c - b$ = lost water „ „ ;
c = rain-water „ „ .

The water lost by percolation is mostly returned at a later date through the springs. As the land area of the Erne basin is about 1,000,000 acres, the ordinates on the diagram show the discharge, or supply, in decimals of a cubic foot per acre per minute. A scale is also added giving the corresponding rainfall in hundredths of an inch per day. The average maximum rainfall for the years 1885–88 occurred in October, followed by August, and then December and January. The loss of water is greatest in August, and gradually diminishes till January, when the springs seem to be fully charged, and little or no loss takes place until April. Evaporation is least in February, and greatest in August. Vegetation begins to affect the loss in February, and attains its maximum in June, gradually decreasing to the end of September. Percolation commences when the influences of vegetation and evaporation diminish sufficiently to allow the rain to pass through the surface-soil, and varies in amount according to the degree of drought of the previous summer. In the fairly wet summer of 1886, the springs were not much exhausted, so that the land was quickly charged again, and by the middle of October, percolation almost ceased ; but in the very dry summer of 1887, the springs were much exhausted, and were not fully charged again till February. The greatest floods take place in December and January ; and the average gradually diminishes to July. October, however, is liable to exceptionally high floods, such as occurred in 1886.

The maximum flood assumed for the Erne was 750,000 cubic feet per minute, while that observed was 730,000 cubic feet per minute. The average summer flow assumed was 100,000 cubic feet per minute, while that observed was 40,000 cubic feet per minute for one year, and 320,000 for three years, making the average 110,000 cubic feet per minute. The diagram shows the duration of flood; for taking the line of 600,000 cubic feet per minute, it can be observed when and for how long the supply exceeded this figure. In February 1886 the excess was 20,000 cubic feet per minute for six days; and, as 1 inch rise per day over the two lakes is equal to 92,000 cubic feet excess per minute, this would cause a rise of $\frac{20}{92}$ inch per day, or a total rise of $1\frac{1}{4}$ inch on the lakes. Again, in October 1886, the excess was 100,000 cubic feet per minute for five days, or sufficient to cause a total rise on the lakes of 6 inches. In January 1887, the excess was 50,000 cubic feet per minute for two days, which would cause a rise of 1 inch. At no other date does the supply-line touch 600,000 cubic feet per minute. As the upper lake may rise 30 inches over the summer level without commencing to flood the land, a pondage is thus provided for floods equivalent to 25,000 cubic feet per minute for thirty days, or 75,000 cubic feet per minute for ten days. At no time did the excess, shown by the line of supply, approach this; so that, a discharging power of 600,000 cubic feet per minute is amply sufficient for all Erne floods. Allowing for the anticipation of floods by the opening of the sluices when heavy rain commences, let it be assumed that a discharging power of not more than 500,000 cubic feet per minute is provided. Following the line of 500,000 cubic feet per minute on January 23rd, 1886, when heavy rain commenced, if the sluices were then opened and a discharge of 500,000 cubic feet per minute was produced, the volume discharged before the flood reached the 500,000 cubic feet per minute, represented on the diagram by the area x, y, z, is equal to the excess represented by y', z', x'; so that for such a flood the lake would be under control (Plate 6, Fig. 2). On the 26th of September, 1886, rain commenced, so that sufficient time was given, by opening the sluices at that date and discharging 500,000 cubic feet per minute, to lower the lakes ready to receive all the excess over the discharging-power that occurred in October from the effect of this rainfall. In a similar way, the excesses of December 1886 and 1887 could have been controlled. Accordingly, with sluices, the results show that the drainage of Lough Erne might have been effected with a discharging-power of 500,000 cubic feet per minute. With a weir, any discharging-power less than 600,000 cubic feet per minute would have been fatal, as on such a system the anticipation of floods is impossible. Again, with a discharging-power of only 400,000 cubic feet per minute, the excess of February 12th, 1885, could have been provided for, by opening the sluices at the commencement of rain on the 22nd of January; and likewise the excesses of February, October, and December 1886. But in January

1887, the time, from the commencement of rain to the rise of the supply-line above 400,000 cubic feet per minute, was not sufficient to prevent flooding. Therefore, with this discharging-power, the drainage would not be effective, unless navigation was so far overlooked as to allow the lakes always to be kept in winter sufficiently below the standard level as to leave a storage equal to the greatest probable excess. For the Erne, the greatest excess, in December 1886, amounted to 100,000 cubic feet per minute for forty days, which would require the lakes to be kept at 45 inches below the standard level, thereby stopping navigation for the winter months. Therefore, it would not be expedient to attempt drainage in the Erne district with a discharging-power less than 500,000 cubic feet per minute (or 0·50 cubic foot per acre per minute), if navigation had to be maintained, and the land kept free from floods.

The natural channels of the Erne could discharge 200,000 cubic feet per minute without injury to the land. In order to discharge 600,000 cubic feet per minute, cuttings were made to provide for an additional discharge of 400,000 cubic feet per minute. If the discharge of 500,000 cubic feet per minute had been considered sufficient, the excavations would have been reduced in the proportion of 4 to 3 = 25 per cent., or say 20 per cent., as certain expenses would have been the same in either case; but it would have been impossible to have obtained sanction for the scheme, if smaller channels had been proposed.

The intervals between the maxima poi[illegible] the lines of rainfall and net supply in the diagram vary [illegible]ation [illegible]e degree of saturation of the subsoil, or the incli[illegible] n diminish [illegible]ce of the ground-water of the district. In summer, the incline being small, the water travels slowly through the rocks and clay into the tributaries, so that the intervals between the maxima points are frequently from fifteen to thirty days. In winter, however, the incline is steep, as the ground-water approaches the surface of the land, and the water flows faster through the ground; so that the intervals between the maxima are sometimes only three or four days. The few instances of long intervals in winter are due to the rain falling as snow, when a specially large proportion of the rainfall goes to the springs.

A diagram like that of Lough Erne may be usefully applied to two purposes:—(*a*) Questions of arterial drainage, where the maxima floods are considered; (*b*) questions of town water-supplies, where the minima points are considered, and the duration of low-water flow is of importance. In either case, the rainfall of the district should be plotted as a diagram to indicate the dates at which rain occurs, and its duration. The lost water may be estimated by considering the character of the district, its geological formation, and the state of vegetation; and deducting this from the rainfall-line, the supply-line of the district is found; or it may be found directly by measuring the discharge of the district at intervals, and plotting these observations in the diagram.

The diagram shows that, from a limestone basin, the yield of the springs for four months in dry summers would be 0·04 cubic foot per acre per minute, and, for a month of this time, it might not exceed 0·03 cubic foot per acre per minute. The rates of rise and fall of the yield of the springs at the ends of the drought are also indicated. For the basalt districts of County Antrim, the dry-weather yield would be only about one-third that of the Erne basin, the limestone of which has an absorbing power about twelve times that of the basalt of Antrim. The absorbing power of the different rocks may thus form a convenient index for obtaining the dry-weather yield of a district as compared with the Erne basin.

In a question of mill-power arising out of a scheme of town water-supply, the Author has made use of the Erne diagram. In many respects the catchment basins of the Erne and the small river in question were similar, so that the diagram was taken as correct for the small district, a new scale being made to read in cubic feet per minute the supply which the diagram would show to be taking place at any time in the smaller district. This small (Ardpattan) district, with an area of 6,780 acres, is within a few miles of the Lough Erne district. Taking into account elevation, rainfall, &c., and using the Lough Erne diagrams, the results obtained did not differ more than 10 per cent. from those actually recorded. Diagrams thus made, from data supplied by rivers under various conditions, would prove very useful in determining what works should be done for the drainage of districts where no such extended observations had been made.

The Erne diagram shows how little the storage in the lakes has to do with summer flow (Plate 6, Fig. 2). The space between the net supply-line and the line of sluice discharge represents storage water; and as these lines nearly always coincide in dry weather, none of the summer flow is made up of storage, for all the storage is wasted before the yield of the springs is reduced by drought; and this occurs in a district where the pondage areas are unusually large. Thus the summer flow of a river is made up of the yield from the springs, and the excess rainfall over 0·15 inch per day, which seems to be the amount of rain taken up by vegetation and evaporation during the summer months.

The general effect of the Erne drainage has been to keep the lakes at a more uniform level than heretofore, maintaining the water-surface higher in summer and lower in winter. The effect of this on the ground water-level is twofold. In flood, the outfall of the ground-water is lower, and the rate of flow of this water through the ground is thus increased as the incline is increased. Accordingly, the percolation-water gets to the lake quicker than formerly; but as the ground is dried to a greater depth, the percolation-water is greater, and the ground-water is less. This has the effect of reducing the maxima points of flood supply, except in cases where much work is carried out on tributaries so as to increase their rate of flow; but, at the same time, it increases the duration of floods, as the percolation-water is added to the storm-water at the end. As less water is left in the soil at the end of winter, the total winter flood is greater, but its maximum lower, and thus more uniform. Less water is also lost by evaporation, as the rain sinks more quickly into the land. There is a reduction in the dry-weather yield, as less water remains stored in the ground at the end of the winter. Again, the dry-weather yield is reduced by the lake being kept higher in summer than formerly, as this prevents the ground-water being drawn off to as low a level as heretofore. The real store of water to keep up the dry-weather yield is the ground-water. Taking the area of the catchment-basin as 25 times that of the lakes, and the absorbing power of the ground as 10 per cent. of its volume, the water stored in the pores of the ground, per foot of depth, is 2·5 times the storage of the lakes for the same depth. Then taking the average level of the district as 50 feet above the lakes, the storage in the ground is 125 times that contained in 1 foot of the lake. This shows that the chief store for the summer yield of a river is the ground-water.

As it is of great importance, in the interest of fisheries and mill-powers, to keep up the dry-weather flow of a river higher, if possible, than it would be in the natural state of things, the Author would suggest that, in all cases of drainage, the free, uncontrolled gap, required by the Fishery Commissioners, should consist of one deep cut, the full depth of the deepest outfall channel, and of such a width that its discharge, at the lowest level allowable without stopping navigation, should be a little more than the natural extreme dry-weather yield, and, when full, that it should discharge the ordinary summer flow after storm-water has passed off. For the former, 0·032 cubic foot per acre per minute might be taken, and for the latter, 0·065 cubic foot per acre per minute. Such a gap would not only draw to the fullest extent on the lake-storage gradually; but also to the same extent, and in the same manner, on the ground-water store, also to its lowest level, when it flows off gradually as wanted.

The extensive flat lands bordering Lough Erne were not only liable to injury from sudden floods, but to the more injurious effects of water remaining on the ground, one flood receding so slowly as to be overtaken by another; so that, during the whole summer of 1879, these lands were covered with from 2 to 4 feet of water. The evil effect of such flooding does not cease with the loss of one year's crop of grass, but the decayed grass becomes so intermingled with the fresh growth of the following year, that the hay of next season is bad in quality, producing diseases in cattle. Large tracts of undrained land affect the agriculture of a wide district, for the spongy grass exposes a much larger surface to evaporation than even a sheet of water; and consequently the air in summer is chilled; and the sun's heat is so wasted in drying up undrained lands, that the mean summer temperature is reduced in many cases by 2° or 3° Fahrenheit, which often makes the difference between cereals ripening or not, and certainly delays the harvests for two or three weeks. The damp chilled air, being mixed with the products of decaying vegetable matter, has made the Lough Erne district notorious for rheumatic complaints.

From the wide-spread benefits derived from arterial drainage, it would be fair that the whole catchment-basin should be included in a rate in aid of large drainage works, not only on account of the benefits the whole valley derives, but on account of the waters which are sent down from the higher lands with greater rapidity, as "thorough" drainage is extended. Great facilities were afforded for a perfect system of drainage by the ponding of the lakes of the Erne. By the retention of part of the floods without injury to the lands, time is given to prepare the lower lake by running it down beforehand, which thus acts as a reservoir to receive the coming flood, which can be anticipated, by telegraph, by at least forty-eight hours. Without such ponding powers, the works would have been nearly doubled in cost. A post-card and occasional telegrams have been found sufficient to apprise the sluice-keeper of coming floods. Having regard to this partial ponding of floods, and the saving so caused, the Author considers that, in rivers without any lakes on their course, it might be often desirable to create artificial lakes in suitable places where the land is not valuable. Such tracts of land, being only occasionally flooded, might be formed into osier-beds, and become a source of profit. Undoubtedly some lands are deteriorated if deprived of floods, such as parts of the "Callows" along the River Shannon; indeed, microscopic examination of some of the flood deposits showed that they contained animalcules which had a manuring effect. The superiority consequently of regulating a river by sluices instead of by fixed weirs, is enhanced, for the power is thus retained of occasionally flooding the flat lands by keeping the sluices shut down when floods occur at suitable times (such as in early spring), and continuing the submersion for a few weeks only.

There are some general principles in designing drainage works which the Author thinks might be adopted with advantage.

(1) The cross section of the excavated channels might have a hyperbolic instead of the usual trapezoidal form, with greatly increased depth in the centre, affording greater discharging capacity for the same area of cross section, obviating slips by the flattened slopes next the land, and confining the channel in times of low-water, thus giving opportunities for cleansing the sides. Such channels would be also more easily excavated, as a dredger could work in the centre during times of low-water. For this last reason alone, it was found necessary to adopt this section for portions of the Lough Erne channel.

(2) The longitudinal section of the improved channels has generally been drawn regardless of certain local conditions, especially where the fall is too much, and the resulting velocity too great for the stability of the sides of the channel, so that they are cut away and the lower reaches silted up. Generally, when the cuttings are through ordinary clay, the channel should be so designed as to form reaches with only 1 foot or 2 feet fall per mile; and, in some cases, to carry out this, embankments become necessary. If the natural fall of the country is greater than necessary, the excess of fall should be accumulated at certain points, and the water discharged into the lower reaches in narrow paved or concreted channels at suitable intervals. Where the system of drainage is by back drains, the river being embanked, these back drains can conveniently be discharged at the foot of these water-slides, thus obviating flap-valves at their point of discharge into the main river. This system is applicable more particularly to small rivers where drainage operations are often a failure from the sides of the channels being cut away and other places silted up. Another advantage of this system of drainage is a saving of expense in cuttings.

The question of confining rivers by embankments, instead of lowering their bed by deep cuttings, is often much discussed. No general rule can be laid down; circumstances and conditions will

APPENDIXES.

I. DREDGING PLANT.

The dredger, 100 feet by 21 feet, and 4 feet 6 inches draught, was driven by a 75 indicated HP. marine compound condensing engine, supplied by a double-flued return tube boiler, with 60 lbs. per square inch working pressure, geared so that 60 revolutions of the engine per minute gave a bucket speed of 40 feet per minute, a friction-wheel being used on the top tumbler shaft. The buckets, twenty-one in number, were of welded steel, riveted to cast-steel sole-links, and were provided with lips of ¾ inch steel plate, the capacity of each bucket being 12 cubic feet. The spare buckets held 8 cubic feet, and were formed of two steel castings (each casting consisting of a combination of a sole-link and bucket side), connected by a bottom and a top plate fitted inside, and riveted to flanges cast on the sides in such a way that no rivet was put in direct strain when the bucket was cutting. The travelling gear consisted of two winches (bow and stern), each working three anchor-chains, driven by friction-gear off a longitudinal shaft geared to the main engine. The lifting gear for the ladder foot was driven by friction-gear off the same longitudinal shaft. The hull was of iron, divided into four water-tight compartments.

The iron barges, 120 feet by 22 feet, with 1 foot 10 inches draught empty, and 4 feet 6 inches loaded, were divided into four water-tight compartments, built the same fore and aft, and provided with bow and stern rudders.

The iron tug, 60 feet by 10 feet, with 4 feet draught forward and 5 feet aft, was divided into two water-tight compartments, propelled by a pair of twin screws, three bladed, 3 feet 6 inches in diameter and 5 feet pitch; the blades were of detachable wrought-iron plates, so as to be easily repaired when injured by striking against boulders in the river-bed. The engines were compound, non-condensing, of 30 indicated HP. each, working at 200 revolutions per minute, and the marine boiler had 80 lbs. per square inch working-pressure.

II. COST AND REMOVAL OF EXCAVATION.

Each shovelman moved about 12 cubic yards per day, which, with 2*s.* 6*d.* per day wages, made the cost of shovelling 2½*d.* per cubic yard; and getting blue clay, three pickmen to one shovelman, cost 3 × 2½ = 7½*d.* per cubic yard. Taking the average lead as two stages of barrows, the net cost would be, getting 7½*d.* + filling 2½ + wheeling 5*d.* = 1*s.* 3*d.*; and adding 20 per cent. for overseeing and use of plant, the cost of removal of the hard clay, exclusive of dams and pumping, was 1*s.* 6*d.* per cubic yard. Half-yard wagons, on 20-lb. rails, laid to a 2-foot gauge, were used, where possible, instead of barrows, to reduce the cost. Where the lift was 25 or 30 feet, and the distance short, an incline was laid at 1 in 5; on this was laid a double road, which was joined in one at the top, and continued level to the tip. At the bottom, each road was run level into the face of the cutting. On the top level, about 15 feet from the incline, a frame was fixed, carrying a 2-foot pulley, with its axle horizontal, at right-angles to the line of way, and about 6 feet above it. Round this pulley a rope or chain passed, one end of which was hooked to the loaded wagon at the bottom, and the other to the empty one at the top of the incline; a horse was hooked to the back of the empty wagon, and pulled it down to the bottom of the incline, at the same time pulling up the loaded wagon. One horse could serve seven shovelmen on a 30-foot lift, making the cost of horsing 0·714*d.* per cubic yard, the horse and driver costing 5*s.* per day, and a shovelman filling 12 cubic yards per day. Sometimes two horses, in tandem, took up two wagons. With heavier horses than those of the district, the inclines should be 1 in 8, with 1 cubic yard wagons.

Where the lift was not so great, and the horizontal distance longer, single roads were laid at 1 in 9, with a gradient of 1 in 50 on the top of the bank up to the tip, so as to run the empties back by gravity. Each horse pulled a wagon from the face, up the incline, to the tip, and so kept three shovelmen at work, the cost of horsing being ⅚*d.* = 1·66*d.* In positions where a number of wagon-roads could be concentrated, a winding-drum, driven by a portable engine, or a steam-winch, was used with inclines of 1 in 5, a double road being used to take up three wagons in the rake, and at the same time let down three empties. The cost of this winding arrangement, for labour, repairs, and coal, was 20*s.* per day, so that no saving could be effected until the work of four horses could be concentrated at one point. Steel-wire ropes, ½ inch in diameter, were used on the drums, those made with the twist of the strand and the rope in the same direction being the most durable.

III. Blasting, Excavating and Dredging.

Explosives were only used for removing clay in specially hard spots; as they did not pay unless the saving of time was important. Dynamite was used as little as possible, on account of the difficulty of getting the workmen to take proper precautions, and from the impossibility, in such a locality, of getting dynamite fresh from the works, as it was found not to keep well, when subject to considerable changes of temperature. For blasting clay, dynamite is not good, as its explosion is too sudden and local; and common blasting-powder is too slow, giving the clay time to yield without cracking. Tonite was found the most suitable, as it allows of hard tamping like powder, and has an explosive action intermediate between powder and dynamite. The cost of blasting clay with tonite was 4*d.* per cubic yard for labour and materials; but the clay was thrown out in such large blocks as to require some picking. For rock, common powder was found the most economical, tin canisters being used in wet holes, the cost ranging from 3*d.* to 6*d.* per cubic yard for labour and materials. In tough, bound corners, the stronger explosives were employed. Electric firing was used with good results, where a number of simultaneous blasts were required along the face; but as much of the rock could be worked out with a crowbar, such close blasting was not often required.

For repairs of plant, larch was preferred to other timber for its lightness and toughness; good trees were delivered on the works at 13*s.* 6*d.* per ton.

Cast-steel hammers were adopted, and steel chisel-pointed picks with hickory handles, ash handles being useless in tough clay. Mild steel wagon wheels were used; for though they cost 60 per cent. more than cast-iron wheels, they never break.

A Chaplin steam-navvy worked in the excavations, under unfavourable conditions, for the 9-foot face was only sufficient to half fill the bucket at each lift; and as the machine had to be worked at the lowest levels, it was frequently flooded. The navvy filled wagons at 6*d.* per cubic yard, in place of 10*d.* by manual labour. The clay in places was so hard that it had to be blasted in front of the navvy.

In dredging, the ladder was mainly kept at the bottom level; the dredger moved from side to side, excavating the full height of the face. But at times a bench 3 feet deep was made to prepare a place for dredging in the summer, as it was expedient then to keep the lake as low as possible to facilitate the dry work. The cost of dredging, including repairs, labour and stores, was 1*s.* 1*d.* per cubic yard for rock and clay together. The clay cost 10*d.* per cubic yard, and the rock 2*s.* per cubic yard, deposited 2 miles off; the items being, for clay, labour 5*d.*, coal 2½*d.*, repairs and stores 2½*d.*; for rock, labour 10*d.*, coal 5*d.*, repairs and stores 9*d.* The rock, being in thin horizontal beds, could be dredged.

The shoots were widened from 2 feet 9 inches to 3 feet 6 inches, to allow the freer discharge of large boulders and blocks of clay. A 4-inch combined centrifugal pump and engine flushed the shoots; and a storage tank over the shoot let down a sudden rush of water to clear away stoppages. A wooden pontoon of 12 tons displacement, put under cantilevers at the front of the dredger, raised the forward part and reduced the plunging in working in hard stuff, so that dredging could be done in 4 feet of water instead of 6 feet.

A compound steel plate proved the best for the bucket lips; the inner half of the lip, being of hard steel, kept a good edge; while the softer material gave toughness, wearing away at the edge so as to leave a thin cutting edge of hard steel.

A special single-chain grab-dredger, patented by the Author, was constructed for removing the dams, the cross-axle being dispensed with to allow of the lifting of large stones, and the head-gear very light, so as to avoid the breakages of the bucket and moving parts caused by the drop of heavy castings. The gearing was high, so as to cut through tough clay without the bucket lifting before being properly filled.

IV. BRIDGE AT ENNISKILLEN.

Across the principal channel at Enniskillen a new stone bridge was built, having two elliptical arches, 70 feet span, 18 feet 6 inches rise, centre pier 8 feet wide, and keystone 2 feet 9 inches deep. The stones for this were cut from the excavation at Belleek, conveyed to the site by water, and set in cement.

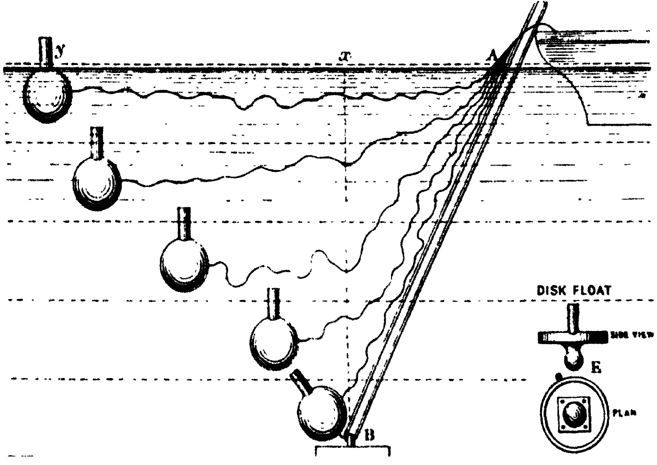
Half of the old bridge, in width, was taken down and the traffic diverted to the remaining half. Half the new bridge was then built alongside; and the traffic was diverted on to the new half bridge, during the building of the remaining half. A proper joint was made between the two sections, by first laying in the centre stones dry, and when both sides were completed, taking them out, and resetting them in cement. The bridge was built in cement mortar, and about ½ inch of settlement took place on striking the centres. The use of cement mortar for arch sheeting almost obviates settlement, and enables a good arch to be made with rough stones. The stone and cement arch has, moreover, the advantage over a cement-concrete arch in not cracking, as a long length of concrete often does. The resistance of the cement mortar under compression is the measure of the breaking-weight of the arch in the former; while in the latter, it is the smaller resistance of the cement concrete.

Discussion.

Mr. James Price, senior, said that the diagram, Plate 6, Fig. 2, required a good deal of study. At any date it was possible to see at once what effect had been produced by the quantity of rain falling over the whole area, and after what period of time it had been produced. The dotted line was not of much consequence; it was a line from which the Author had made the other calculations; from the opening of the sluices or from the flow of water, or from both, together with the rainfall, he had been able to ascertain the line which was really important—the net supply that came into the lake, which was what had to be dealt with in all drainage operations. If a line were drawn joining the lowest point of each of the dips it might be called the spring-water line, as it would not be affected by recent rains. With regard to the measurement of the flow of water, he believed no accurate nor reliable method of ascertaining the flow in a river existed. He had been trying a system that he thought would give the nearest approach to accuracy, having also the advantage that the experiment could be repeated many times under the same circumstances. No length of river was needed, one accurate section being sufficient. A ball, about 8 inches in diameter (*Fig. 1*), was weighted so that it would just float, and a boat was anchored a little above the position; the ball was attached by a slight thread to an endless

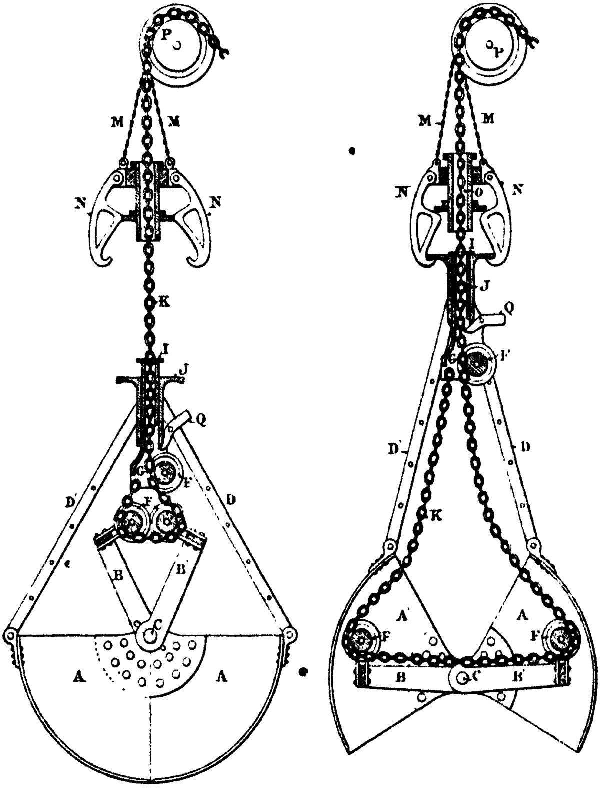
Mr. Price, sen

Fig. 1.



Price, sen. cord passing through a pulley attached to a heavy weight on the bottom of the channel. The ball having been drawn to the bottom, at an observed time the thread was broken, and the ball thereby released. There was a small period of time lost in the ball attaining uniform velocity; but after a very short distance (which he was trying to ascertain accurately) it did attain a uniform velocity of ascent. It was quite evident that the ball in ascending to the surface passed through each different stratum of water: the time of ascent gave the mean velocity of the current. Of course there were other methods by which the ball could be released from the bottom. A frame of light timber was allowed to float on the water moored to the boat by a string; this was

Figs. 2.



CROSS SECTION SHUT
POSITION No 1

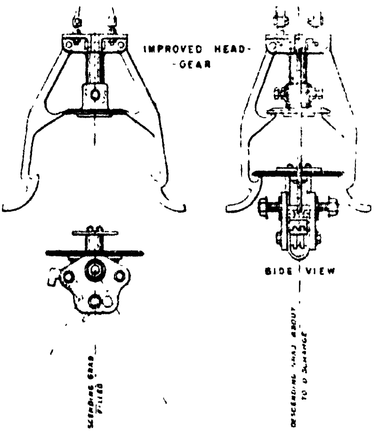
CROSS SECTION OPEN
POSITION No 2

Scale $\frac{3}{8}$ inch = 1 foot.

Mr. Price,

drawn up-stream or allowed to float down, so that it might mark the place of appearance of the ball; thus the experiment could be repeated several times and the average distance taken. This was done through the whole cross section of the river. Owing to the absence of his son, he had not the advantage of his assistance in carrying out these experiments, but he hoped to complete them. The difficulty of measuring with weighted rods was well known. Even if they went close to the bottom they did not give the mean velocity. That method also required a long reach of uniform channel, and gave a great deal of trouble. But by his system a person could sit in a boat, cross the river from one side to the other, and get an accurate measurement of the mean velocity.

Figs. 3.



Scale ⅜ inch = 1 foot.

Another form of float, namely a disk weighted underneath, was perhaps better than the ball, being more easily made, and acquiring its uniform velocity of ascent sooner than the ball. The grab had been used with great effect. It was different from any other (*Figs. 2* and *Figs. 3*). The whole inside of the grab was free. It was very useful for taking up large stones and boulders. There were only three pulleys, connected with levers which closed. The principle was a rather curious one. The grab had a considerable gripping force; it worked with a single chain. The most ingenious part of it was the arrangement whereby the grab when it came up was gripped by the claws. The sluices had worked effectively, they were working still, and had never given the

Mr. Price, sen. slightest trouble, nor cost anything for repair. The lake district was a remarkable one. He did not know any other consisting of so long a tableland. It formed a sheet of water 52 miles long with a fall of only 6 inches. He considered it unique in the British Isles. It was now a very fine navigation; but its commercial value was unascertained, as its use would depend on circumstances not connected with Lough Erne.

Sir John Coode. Sir JOHN COODE, President, said that two references were made in the Paper to the inclination of the ground-water. If the Author could give the rates of inclination through the different strata, it would materially add to the value of the Paper. It was very rarely that such observations had been made; there was a general deficiency of accurate data upon that point, and if Mr. Price or his son had any observations upon it, it would be very undesirable that they should be lost.

Mr. Stoney. Mr. F. G. M. STONEY wished to make a few remarks on some of the minor details in connection with the sluices. He hoped to write a Paper upon larger sluices, including those of Lough Erne, and would, therefore, not anticipate too much. The Author had stated that the sluices worked almost as easily down at the bottom of the water with a full head on as they did in air. Mr. Stoney wished to make an addition to that statement. He constructed a little wooden handle of only 6 inches radius; with this he was able to start and lift the sluice under the abnormal head of 16 feet, that was 1 foot 6 inches going over the top of the gate, and he could not detect the slightest difference in starting the sluice, with all that head, from the power required to move it in the air. There was a curious confirmation of that statement. He had spent a fortnight in calculating the comparative pitches of screws so as to arrive at a pitch that would allow the gates to creep down, and not to race. It seemed very easy, but it was not so. He considered himself somewhat fortunate in arriving at the results that had been obtained, because the sluices weighed 13 tons each, and there was no counterbalance. To begin with, he almost eliminated friction and resistance at the head of the screw, but could not get over the resistance in the nut. The nut was very long, and the pitch of necessity was considerable. Out of four sluices two crept down very slowly. One of them required the least occasional touch on the main spur-wheel after the pinion was withdrawn; in the case of the other, the man in charge occasionally put a hammer handle against it as it was inclined to race somewhat. As a matter of fact, the quality of the lubricating oil had more effect on the gear than all the water-pressure. The interesting engineering point about it was this. He had started the sluice to go down when a full flood was passing through; the sluice-door was open about 8 feet 6 inches or 9 feet, and there was a depth of 7 feet 6 inches or 8 feet of water passing through at high velocity. It was a wide river with a fairly sharp fall and a direct straight approach, and a head of 14 feet at the upper end, so that apart from the bottom-velocity there was a considerably increased surface-drop in the water approaching the sluice. The sluice commenced to go down slowly like clock-work. The wheels could be seen going round pretty fast, but without being close to the door its motion could not be noticed. The lower edge touched the water passing through, and was not retarded in the slightest degree, but went to the bottom without any diminution in velocity. He could not understand this for a long time; but, watching the sluices from day to day, he found out the cause. From actual experiments he had made on a large scale, he had no doubt the traction on the main rollers would not exceed some 250 to 300 lbs., and the door creeping down slowly with apparently no reserve of power would gradually have to overcome this traction, increasing from nothing at the top to a few hundred lbs. at the bottom. Close observation plainly showed a positive vertical movement of water down the skin of the gate, the friction of which, combined with a certain enlivening action of the flow, seemed to fully compensate for the increased traction, but inasmuch as this entire force was only some few hundred lbs., its effect was not perceptible on the lifting-gear, as the gate moved only $\frac{1}{24}$ inch for each turn of the handle, and the real work to be done was that of lifting the 13 tons of gate, the traction being practically nothing. Another proof of the skin action was that the last line of rivets on the bottom of the doors was 3 inches from the lower edge, and when the doors were open about 5 feet, with about 11 feet of water upon them, he had seen unbroken water for 50 feet below of dark, clear water, and the whole surface was streaked with marks, each mark corresponding to a rivet line, showing the quantity of water which made its way down by the skin of the gate. The Author was the first engineer in Great Britain who had had the courage to use things of that kind, which could not be tried experimentally. The Author had said to him, "we cannot open these big things as fast as we want; we must have a turbine." Mr. Stoney was afraid to use a turbine, because he knew that with a machine of that kind running at such a high velocity, with nothing apparently to control it, if there was a check anywhere, it would be smashed to bits. He therefore devised an automatic release. Upon one end Mr. Ston

Stoney. of the rocking-lever there was a cast-iron weight 5 inches in diameter and 7 inches long, which was quite sufficient to actuate one of the sluices. He made it light for the purpose of transmitting the least possible excess of power beyond what was actually required, so that if a serious check took place, the friction would not drive against any abnormal resistance. But apart from that, he depended from one end of the rocking-lever a small iron rod. When the gate came up it lifted this rod, eased off the weight, and so formed an automatic release. On the other hand, he dropped a long rod from the opposite end of the rocking-lever, having a stop at its lower end, which passed through the top member of the gate, and when the gate came within a few inches of the bottom, it pulled the lever and disengaged the weight as before. When the gate reached near the top or near the bottom, the turbine and shafting ran on, but no motion was conveyed to the gear, and the gate remained still. No accident had ever taken place, by overwinding. With regard to the coefficient of discharge, he thought if the Author had had sufficient water for testing the four sluices he would have obtained a better result. It would be seen from the section of the pier, that there was a very long sharp cut-water. If any one of the sluices was opened the water would not approach the sluice in straight lines. In the early days of working the sluices, he had a great fright in opening one sluice under a full head of water. The place was dry on the down-stream side of the sluices, and the water was about coming over the top. He put the turbine on and lifted the sluice as quickly as he could, and then he thought that the concrete bottom was swept out. A tremendous eddy was formed round the projecting nose of the cut-water. In that way the efficiency was reduced. He might be permitted to add that in putting the sluices up, he received a lesson in sluice-building. A land commissioner saw them, and was struck with their weakness. He saw that the skin was ½ inch thick, and he spoke to Mr. Stoney on the subject. As a matter of fact the door was 7 feet thick. He said, "Are you not making the doors very weak?" Mr. Stoney replied, "I was informed that I had to hold up a head of 14 feet 6 inches of water against the sluices, and I think they will hold it." "But, my dear sir," said the commissioner, "you have 52 miles of water!" Evidently the commissioner was not familiar with the hydrostatic paradox.

Hassard. Mr. R. HASSARD said that the history of the Lough Erne drainage was a rather curious one. The question of the construction of works for the relief of the periodically inundated lands bordering on Lough Erne, especially upon the upper lake, had been suggested from time to time and agitated during the last fifty years. The first feasible plan brought forward was a complete one; it was introduced in 1847 by Mr. Forsyth, who was for many years connected with the Board of Public Works in Ireland. His estimate for the works was £156,000, and it was very similar in all respects to the project as carried out by Mr. Price, except in the omission of a lock and weir, with a small fall of 2 feet between the lakes, which the Board of Public Works thought necessary at first for the maintenance of the navigation. But at that time the estimate was not thought reliable, owing to the cost of several works carried out by the Board of Works having been largely exceeded, although carried out under their own officers. The project accordingly dropped through, and was not revived for several years, when Mr. Porter, a gentleman resident in the locality, and a large landowner, considerably interested in the matter, re-commenced the agitation for the resuscitation of the project, and it was entirely owing to his energy and perseverance that it had been carried out. That was in 1873. Mr. Porter employed Mr. Lynam, well known in Ireland as an engineer, to make an estimate. The late Lord Erne employed Mr. James Barton, of Dundalk, also well known in Ireland, to investigate the matter. Mr. Lynam's estimate was £168,000, and Mr. Barton's £169,500. Mr. Lynam's connection with Mr. Porter shortly afterwards ceased, and the matter was placed in Mr. Hassard's hands, and he was asked to make an independent estimate, which he did, amounting to £172,000. It would be seen that the three estimates were approximate, within a few thousands, showing, he thought, the reliability and care with which they were framed. But a sum of £170,000 was thought more than the locality could bear, and the matter again dropped. About five years afterwards Mr. Porter, who was indefatigable, re-commenced his agitation. He wished Mr. Hassard to act for him, but after a short interval, he ceased to act, and Mr. Porter placed the matter in the hands of Mr. Price, who issued a pamphlet, in which he stated that a complete scheme of drainage could be carried out for £68,000. It was no doubt very gratifying to the landowners to be told that they could have carried out for that amount what preceding engineers had valued at £170,000, and that set the ball rolling again. Soon afterwards Mr. Hawksley, Past President, was instructed by the Board of Works in Ireland to hold an inquiry at Enniskillen, and he did so. At that time Mr. Price's estimate had reached £94,000, and Mr. Hawksley added another £10,000, making it £104,000. Mr. Price

Mr. Hass

r. Hassard. had stated that the works had cost £180,000, but he had not explained the reason for the great difference. He had partially alluded to the large compensations paid to the Fishery Company. Mr. Hassard knew what the amount of those compensations was, having been employed by the Fishery Company. They amounted to very little more than it was well known in the first instance would have to be paid. As to the works themselves, it had no doubt occurred to many members to ask why the machinery and the appliances for controlling the water in the lakes were not placed at the outlet, instead of in a river about 4 miles distant. There was no engineering reason why the sluices should not have been situated at the outlet. The water was shallow, the foundations were excellent, and the sluices, or a combination of weir and sluices, might have been placed there without any difficulty; but the Board of Works had a very high estimate of the value of Lough Erne as an inland navigation, and thought that if the navigable channel were extended to 4 miles below, to a place called Belleek, such advantages would be conferred upon the locality that it should contribute towards the cost of construction, and placing the sluices at Belleek was made a peg on which to hang a contribution of £30,000 from the immediate locality. He thought that under such circumstances the Drainage Board was right in putting the sluices where they now were. With regard to the channel from the outlet of the lake down to Belleek, in the year 1873 he was called in, as he had been on several occasions since, by the Drainage Board to advise on various points that had cropped up from time to time. At that time he found that the dredger, by means of which it was intended that the channel, 4 miles in length, should be entirely excavated, was turning out only about 200 cubic yards per day. At that rate of progress, making allowance for the time that the dredger would be necessarily idle, from renewing the bucket-lips or the breaking down of the machinery, it would have taken fifteen years to complete the work, and the dredger was required in other parts as well as in that particular locality. It therefore occurred to him that the shape of the channel might be changed. The outlet of the lake was a broad, shallow stream; and if dams or stanks could be made and the dredging confined to the central portion of the channel, the two sides could be taken out by manual labour; and instead of the excavation proceeding only at one spot, the works might be prosecuted almost simultaneously over nearly the whole of the channel between the lake and Belleek. That process was adopted, and the greater part of the work was done by manual labour—

which was the means of giving a very large amount of employment in a locality where it was very much needed; the work being in the neighbourhood of one of those poor and congested districts of Ireland of which so much had lately been heard. The sluices no doubt acted admirably; but he thought it would have been wise, in the construction of the work, to have provided some means of isolating each sluice chamber, so that if any accident happened, either to the floor or to the sides of the work, the water could be excluded for a time to allow the necessary repairs to be effected. Mr. Stoney had said that he had lifted, by means of the turbine, one of those sluices rather rapidly, and that there was such a rush of water through the channel that he thought the floor of the chamber behind the sluice would have been blown out. Another thing that he thought might have been done with advantage was the lengthening of the piers somewhat upstream, making semi-circular instead of pointed ends. The water, as Mr. Stoney had stated, was much disturbed when one sluice was open and the adjoining one shut. He had himself seen, where the water was trying to get round the point of the cut-water, a whirlpool about 2 feet in diameter and 5 feet deep, down which a stick might have been put without touching the water. He also thought that the lengthening of the piers would have admitted of fender-girders being placed across the channel in front of the sluices, because it was manifest that a lumber boat or anything of that kind getting adrift in the lake above might, on going down at night, do serious damage to them if some protection of that kind was not adopted. In accordance with his advice, channels or grooves were made in the cut-waters, and beams of wood placed across. It would however have been better if iron had been used in the first instance. Mr. Hassard.

Mr. J. W. Barry thought that the members would admit the great importance of the subject of arterial drainage to engineers; and having had some special experience of the matter in Ireland during the last few years, he might, perhaps, offer a few remarks on the Paper. The importance of arterial drainage in Ireland was unusually great, owing to the remarkable configuration of the country. The general description of the land in Ireland was that of a slightly elevated plain extending over a great part of the country, which was surrounded by a rampart more or less continuous of limestone, of which material the plain itself was mainly composed. The result of that state of things was that most of the main rivers in Ireland had a remarkably flat course for many miles from their source, and a very precipitous course for the few Mr. Barry.

Mr. Barry. remaining miles that enabled them to reach the sea; and the problem which the engineer had to face in Ireland was to improve the flow of water over the long length of river course in the plain, and then allow it to discharge freely down the more precipitous and shorter course to the sea. That was really the key of almost all the large river improvements which had been so much talked of in Ireland. The importance of effective arterial drainage to the country consisted, mainly, in the agricultural pursuits which such a considerable proportion of the population pursued, making all matters affecting agriculture of far greater relative importance in Ireland than in England. He had found that much difference of opinion existed in many localities as to the value of excluding floods from land, or of letting them flow over the land at certain times of the year; and some persons had been disposed to assign a large amount of value to the fertilizing properties of the mud left by the floods. The Royal Commissioners of Irish Public Works came to the conclusion that although there was some value from the deposit in certain localities, yet, on the whole, the benefits of improving arterial drainage and diminishing floods were undoubted, not merely from an agricultural point of view, but from the points of view of the general health of the neighbourhood, and of the question of temperature alluded to by the Author. The manurial value of the mud left by the floods was very largely discounted by the fact that they induced a much coarser kind of grass to grow upon the flooded lands, and it was impossible to produce the best kind of grass for hay unless the floods were excluded. That, he believed, was now becoming, to a great extent, an accepted axiom in Ireland. These being the facts of the case, and arterial drainage being of such great value in Ireland, Government had, for many years, looked with a most favourable eye upon all schemes of improving arterial drainage, and had spent or assisted the expenditure of very large sums of money. In many cases that expenditure had been wholly contributed by the nation, but in others a large proportion of it had been found by the landowners and others interested in the various localities. Government also, in times of scarcity, had spent large funds which were raised for the relief of distress on the laudable enterprise of improving arterial drainage, while at the same time feeding the starving population. Some of those works were done under pressure of time arising from the necessity of finding work quickly for people. Mistakes had consequently been made in Ireland, and some of the works that had been undertaken had proved very expensive having regard to the results which were attained. But he thought that, on the whole, it might be said **Mr. Barry.** with great certainty that the action of Government in Ireland, in the matter of arterial drainage, had been decidedly beneficial, and that very important works, which had resulted in great good, could never have been undertaken unless Government had taken the position of not only finding money, but also supervising the works. The Rivers Bann, Shannon, Suck, and Lough Erne were all examples of large works which had been helped by Government, and which, although not entirely successful, had produced valuable results for the country at large. Last year Government was prepared to spend from £300,000 to £400,000 in improving the drainage of the Shannon, the Bann, and the Barrow; but unfortunately, owing to circumstances over which the Government could not perhaps exercise much control, the Bills never arrived at maturity, and those works which were urgently required were consequently still in abeyance; he hoped that when the time was more opportune the money would be forthcoming, and the works would be undertaken. One matter referred to in the Paper was of considerable importance as affecting one of the rivers which the Government proposed to deal with, namely the Barrow; he referred to the question of embanking the river, as compared with dredging or deepening it for the whole of its course through a flat plain until it reached the more precipitous part below Carlow. In Ireland up to the present time, with perhaps two exceptions, he thought the usual practice had been to deepen rivers, and in doing so, owing to the existence of such an expanse of limestone forming their beds, vast sums of money had been spent, which could have been avoided if a more liberal use had been made of embankments such as were seen in the Fens of England, and of which examples existed in the north of Ireland. The notion which he heard expressed in Ireland was that, if a river was embanked, the people were so careless by nature that they would allow the embankments to go to destruction or decay, and would not maintain them with the care which was necessary for works of this kind. For himself, he did not believe that the works would be neglected. Certainly the rivers he had seen in Ireland that had been embanked were as well maintained as any rivers in England. There could be little doubt that a system of embanking, in the case of the River Barrow, would result in much economy in regard to the first cost, and, as far as he could see, without loss of efficiency in the carrying power of the river. A combination of deepening and embanking seemed to be a most advantageous thing in Ireland, as he had no doubt that it was

Mr. Barry. elsewhere. The best system consisted in taking advantage of the configuration of the land, in applying embankments in places where excavations would be abnormally expensive, and in applying excavation exclusively where it could be done with a reasonable amount of cheapness. A matter which had been found very important in Ireland was the necessity of some new legislation with regard to all matters of river regulation. Numerous Acts of Parliament had been passed, but the present condition of the law was chaotic, and required, in his judgment, most material alteration and consolidation. Such was the condition of the law in Ireland, where some attempt had been made to deal with the subject from a general point of view, but it was still more the case in England. He thought that no one, who had anything to do with the subject of arterial drainage in England, could come to any other conclusion, than that some more drastic and general mode of dealing with all the conflicting interests, which were evoked by any scheme for arterial drainage, was urgently needed for the regulation of the chief river courses of the country. No one could travel through the flat parts of England, in rainy weather, without feeling ashamed that he lived under a system of laws which allowed of such vast areas of land being left to be flooded whenever the weather became moderately severe. The time ought not to be far distant when floods should be brought under proper regulation. With reference to Lough Erne itself, he had the pleasure of seeing a great deal of what had been carried out, and apart from any question of cost, which had been discussed by Mr. Hassard, he thought the work was of national importance, and was a very creditable undertaking. In the Lough Erne scheme would be found the old antagonism between navigation, fisheries, and arterial drainage. There could be no question that the works on Lough Erne had been materially increased in cost and difficulty by the necessity, or the assumed necessity, for dealing very tenderly with navigation interests. The result of the investigations of the Royal Commissioners in Ireland, had been to convince them that the value of navigation facilities had been generally much over-rated, and he could not help thinking that, in the work which they had under consideration, if the question of navigation had been altogether set aside, no one would have been much the worse, but many people would have been considerably the richer. The cost of the works had also been largely increased by financial and legal difficulties, and he could understand that large additions to the original estimates were not unreasonable if those circumstances were

Mr. Barry.

allowed for. Still, even with the large cost incurred, the works were not altogether unsatisfactory in a financial point of view, and from an engineering point of view he thought they were exceedingly creditable. He much regretted that he had not had time to visit the sluices, as he should have been glad to see those important works; but he saw other sluices in Ireland of the same description, not, however, of the same size, and had come to the conclusion that they were the most desirable mode of dealing with flood-water. The sluices at Belleek seemed to him rather peculiar with regard to the form of the piers, and he could quite understand what Mr. Hassard had stated, namely, that large whirlpools and eddies were caused by the very short cut-waters which formed the approach to the sluices and by the swelling outwards of the piers behind the sluices. The great advantage of these handy and easily-worked sluices was that relief could be given to the river quickly, and at a time when that relief was of the greatest value. He was much struck with the investigation which the Author had made into the amount of water which would be given off into the river course at different times of rainfall, and under different conditions of soil. Such investigations were of the greatest value, and it was to be regretted that more accurate records were not kept in all parts of the country of the effect of heavy rains, or even ordinary rains, falling upon different soils on different declivities. It was extremely difficult to arrive at any sound conclusions on these points when the necessity arose for designing works of arterial drainage; and any conclusions based upon a short investigation were likely to lead either to extravagance on the one side or to undue parsimony on the other. If some general means could be adopted of recording such matters, they would be of the highest value to engineers and to the whole community. He had nothing to add but to express his congratulation to Mr. Price on the conclusion of his arduous work, and to say that he thought the Paper was one of those which afforded valuable records on a matter which it did not fall to the lot of many persons to be able to investigate with the advantages which the Author had possessed.

Mr. Birch.

Mr. R. W. Peregrine Birch enquired whether, and if so, to what extent, the commercial results had justified the expenditure of £8 12*s*. 6*d*. per acre, and also whether the Author had any health returns bearing on the subject. He had no doubt, as Mr. Barry had stated, that the health of the district had been much improved by the reduction of the water-level. It would be well if the annual rainfall was stated in the Paper, and if, for the benefit of

Mr. Birch. members not familiar with the local conditions, a cross section or two of the valley were also appended, so that they might be able to estimate the extent of the soakage from which the 3,000 or 4,000 acres had been saved. He was glad to hear the Author's statement that the improvement of the drainage had mitigated the floods. He believed that that was not a generally-accepted view. Floods in rivers were constantly attributed to improved drainage. He had himself always believed that the tendency of improved drainage must be to reduce floods, and he knew from records he possessed that the floods in the Thames had not been higher of late years than they were seventy years ago. The flood in the Thames in 1821 was at least a foot higher than that in 1877—the biggest flood of late years; and the floods of 1823 and 1852 were also higher than that of 1877, although not so high as that of 1821. He had no doubt that more suffering was now caused by floods because a large extent of low land had been built upon during the last twenty or thirty years, which fifty years ago was simply meadows. Referring to the storage of water available for summer flow, he thought the Author had made a mistake in his statement that: "Taking the area of the catchment-basin as 25 times that of the lakes, and the absorbing power of the ground as 10 per cent. of its volume, the water stored in the pores of the ground, per foot of depth, is 2·5 times the storage of the lakes for the same depth. Then taking the average level of the district as 50 feet above the lakes, the storage in the ground is 125 times that contained in 1 foot of the lake." That required some little explanation. It could only be true if by the level of the district the Author meant the sub-soil water-level of the district, and it was only true if he knew that there was no impervious bed rising above the level of the lake. The mistake would be seen when he pointed out that 10 per cent. of 50 feet meant 60 inches, whereas only 30 or 40 inches of rain fell upon the watershed during the whole twelve months.

Mr. Vernon-Harcourt. Mr. L. F. VERNON-HARCOURT said it was about ten years since he had advocated before the Institution the importance of introducing movable weirs for combining drainage and navigation,[1] and he had also had the opportunity of visiting the sluices at Belleek. It was well known by people who had lived, like himself, in Ireland, and also by people who had visited the country, that it was a remarkably wet country, with large tracts of lakes, besides the very indented shores of the west coast. Therefore it might be

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[1] Minutes of Proceedings Inst. C.E., vol. lx. p. 24.

Mr. Vernon-Harcourt.

expected that considerable difficulties would be found in the drainage of flat districts there. Moreover, in many parts there were large areas of bogland, which were excavated to some depth for obtaining peat for fuel, so that large quantities of low-lying land had water standing on them in the centre of Ireland. With regard to what Mr. Birch had said about the extension of arterial drainage diminishing instead of increasing the floods, the Paper showed that that was not the view held by the Author, because he considered that it was all the more necessary to have these drainage works on account of the extension of arterial drainage. With regard to the Lough Erne district itself, it was remarkable that there was a very good fall at Belleek and below. The river below Belleek was more like a mountain stream than a stream near the sea, whereas above there was a long tract of flat land; and therefore it was possible to get a remarkably good fall at the outlet below the lakes. Mr. Price also had the advantage of the large water area of the Lough Erne district for a reservoir for the floods. He thought, looking at the plan of the land surrounding Lough Erne, that, but for the navigation, it would have been desirable to lower the level much more than had been done, because there were considerable tracts surrounding the lake which seemed to be only a kind of marshland; and it would appear desirable to lower that level and gain more land for the purpose of agriculture if that land was of any value agriculturally. Drainage works had been carried out, with great advantage both to agriculture and health, at the lakes Fucino and Copais. The Author had mentioned that Mr. Price, senior, had been to see the various movable weirs in France and other places, and he quite agreed that weirs of that kind, which were very suitable for navigable rivers, would not have been suitable at Belleek, because they were simply required to be raised and lowered at certain periods of the year, and did not adapt themselves so readily to the exact adjustment of the discharge as sluice-gates. The sluice-gates themselves, with a head of 14 feet 6 inches of water against them, could be raised with great ease by turning with one hand the handle of the sluice-gate gear; and it simply required time to raise the gates. The other sluice-gates, of similar type, which had been erected in Ireland on the Suck Drainage, at Ballinasloe, were rather different. He had had the opportunity of inspecting them. They were counterbalanced, and therefore they were moved much more quickly, and rose much more readily; but they were not such large gates as those at Belleek. Similar gates might now be seen in course of erection on the Manchester

Mr. Vernon-Harcourt. Ship-Canal Works, across the River Weaver and higher up. The Author had mentioned that a current meter was not used for measuring the discharge. He had himself found that a current meter, if there was not too sluggish a stream, was very suitable for measuring the discharge of streams, and it seemed a pity that that method of measuring had not been tried as well as others. With regard to the diagrams, he agreed fully with Mr. Barry that records of the hydrology of the district were very valuable. They showed the amount of the supply, and the water flowing into Lough Erne; they also showed the rainfall at each period; and in that way it could be judged how far the rainfall affected the supply in the different months of the year. It also showed what was very well known, but it was interesting to see it reproduced in the diagram, that in the summer months, July, August, and September, the rainfall had comparatively little effect upon the discharge of the streams. In some years the rainfall, if it occurred in July and August, seemed to have no effect upon the supply, showing that the whole of the rainfall, even though it amounted to a fair quantity, was taken off by evaporation and percolation. Apparently, from the Author's diagrams, the greatest rainfall from 1885 to 1888 occurred in the month of July, and it was followed only by a very moderate flood. In some cases the rainfall and the flooding apparently coincided; and in other cases, which was the more natural order, the flood followed the rainfall. He thought that diagrams were most valuable for the purpose of judging exactly how the rainfall affected the discharge of rivers. The Author had suggested that the whole river basin should be rated. That would be a most useful piece of legislation, if it could be carried out, for the purpose of the drainage of a district; because the great difficulty in all those matters in England, as well as in Ireland, was to obtain sufficient money for carrying out works for improving the flooded districts. If a rate was only imposed upon the flooded area, that area was comparatively so small that it was impossible, especially at the present time of agricultural depression, to do sufficient work to remedy the evil. But the improvement, which was effected by the removal of floods from the lower districts, really did improve the sanitary condition of the upper districts; and therefore it would seem fair that a rate of some kind, though smaller, should be imposed upon the whole catchment basin, as suggested by the Author. He had further suggested that reservoirs should be made for preventing floods. That would seem inexpedient, especially in the case of Ireland, where the Author complained of too much water,

Mr. Vernon-Harcourt.

and also where land was well cultivated. There was a difficulty in getting proper places for forming such reservoirs at a moderate cost. It was very well in a case where there was a large natural reservoir like Lough Erne to utilize; but the forming of reservoirs for that purpose was another question. The Author also suggested that, in Ireland, too large a fall had been generally given for the drains. That must be due to the large fall that was obtainable towards the sea coast. It was quite the contrary in the Fen Districts in England, where only a fall of about 4 inches per mile could be obtained; and most probably the reason why so much fall had been given was that it was cheaper to utilize a large fall, and so to reduce the cross section of channel required for a given discharge. The Author had suggested the use of embankments instead of enlarged channels. There was no doubt that a moderate use of embankments might be advantageous, especially where small embankments would provide against the flooding of a small quantity of land. The only objection to embankments seemed to him to be that their use might lead to the neglect of the channel itself, and thus sometimes lead to serious floods from the breaking of those embankments.

Mr. Symons.

Mr. G. J. Symons said he was glad to find that Mr. Barry approved of the Paper with regard to its engineering portion. He was sorry, however, to differ from Mr. Barry, and also from Mr. Vernon-Harcourt, as to the value of the diagrams. He yielded to none in the desire for information of that kind, but he should endeavour to show that the diagram as it stood was not such a one as ought to be brought before the Institution in the year 1890. He thought it very strange that a Paper on the drainage of a district should not contain a map of the watershed. It was true that the details of Lough Erne itself were given, but there was no information as to the watershed beyond a rough statement of the total area. He was sufficiently acquainted with the watershed to know that it had a very different rainfall in different parts. The Author stated that "For several years two rain-gauges have been kept as the standards of the Erne district, one at Florence-Court, on the high ground south of Enniskillen, the other at Colebrook, about 8 miles north-east of Enniskillen." He might state as a matter of history that one of these gauges was established in 1844, and was therefore not put up in connection with the Lough Erne Drainage Works. The Author had taken these two records, but there were four stations within the watershed, and two others on the immediate confines which threw considerable light on the rainfall on the watershed.

Mr. Symons. The two stations that he had chosen gave for the years 1885–1888 an average of 45·5 inches and 38·9 inches respectively, so that taking those two stations, the average of the Lough Erne watershed was 42 inches. But the station at Florence-Court was immediately under the hills on the western side, and the rainfall there was 6½ inches more than at any other station in the watershed. The consequence was that by adopting those two instead of all four, or all six, the Author had an average quantity to deal with of 42 inches instead of 37 inches, which was probably much nearer the truth. Having an extra large rainfall, and then having measured the outflow from the ground, the Author had put down the difference between those two quantities as loss. He had been much puzzled at that, because in one part of the Paper the Author referred to the loss as 0·15 inch per day. He had stated, "Thus the summer flow of a river is made up of the yield from the springs, and the excess rainfall over 0·15 inch per day, which seems to be the amount of rain taken up by vegetation and evaporation during the summer months." The 0·15 inch daily would give 18 inches loss in the four summer months alone, whereas it was well known in England that the loss, at the very outside, might be put at 15 or 16 inches for the whole year. In

—	Edgworthstown, Currygrane.	Lawderdale, co. Leitrim.	Belturbet, Red Hills.	Rockcorry, Monaghan.	Enniskillen, Florence Court	Brookeboro', Colebrook Park.
	Feet Ins	Feet Ins	Feet Ins	Feet Ins.	Feet Ins.	Feet Ins.
Diameter of gauge . .	0 5	0 5	0 5	0 5	0 8	0 0
Height above ground	1 0	1 0	0 9	1 0	1 9	1 0
Altitude above O D .	265 0	275 0	208 0	290 0	250 0	279 0
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.
1885	29·29	36·61	29·77	29·48	50·54	38·40
1886	41·25	44·89	39·96	39·23	48·48	45·08
1887	26·00	33·16	24·50	22·09	35·29	31·46
1888	36·96	41·31	66·15	(33·50)?	47·79	40·69
Means	33·38	38·99	32·59	(31·33)?	45·52	38·91
Mean of two stations quoted by Mr. Price					42·22	
Mean of four stations in the watershed			37·09			
Mean of six stations in or near the						

Mr. Symo

the present instance, however, it might be seen where the Author had got his large amount of loss. He had miscalculated the rainfall to begin with, and therefore a large estimated loss was the only way to make the quantities balance. One error was apt to lead to another. The diagram itself, was rather puzzling; he could not make out whether the rainfall was supposed to be given for days or weeks, or for variable periods. It was represented by a gently flowing curve. The proper way of showing rainfall on a diagram of that sort would be by what was known as the organ-pipe arrangement, vertical columns the lengths of which were proportionate to the quantity of rain falling each day. There were also some other puzzling things about the diagram. Ireland was well known to be wet, but the diagram represented only ten days without rain in 1885, thirteen in 1886, five in 1887, and twenty-five in 1888. It was well known that often for a week together, if not for a fortnight, there was no rain at all, and during that time the line ought to run along at zero, which it did not. If the Author had given the numerical values from which the diagrams were constructed, the members would have been in a much better position. At p. 84 there was apparently a little slip; the Author had stated that the evaporation was least in February, and greatest in August. He presumed that the Author meant the amount of loss. Of course the term loss as understood by engineers contained other things besides evaporation, but with regard to evaporation itself he did not quite see why it should be at a different period in Ireland from what it was in England, and certainly the maximum evaporation in England was in June and July, and not in August, while the minimum was almost always in December and January, and not in February, unless the smaller number of days in that month had something to do with the result. The Author had referred to the useful applications that might be made of diagrams of that sort, as to which there could be no doubt. The suggestion that had been thrown out, that something might be done towards collecting data of that kind was of the highest importance, and if the Institution could see its way to do something of the sort it would prove of the greatest value. But first-rate observations would be necessary. It would be convenient to those who were only on the outskirts of engineering if the same expressions were always used. The Author, in referring to the yield of the springs, gave the number of cubic feet per acre per minute, whereas in England it was usual to give it per cubic foot per second per thousand acres; it would be very convenient if a uniform method were adopted. The question of

Mr. Symons. the influence of drainage on the floods, and the relative heights of floods, as compared with those of past years, was far too large a question to discuss in connection with the present Paper; but it was only right to point out that the Author appeared to contradict himself. He referred on the first page to "the increase of flood-water, owing to the reduction of absorption, resulting from the diminution of timber since the year 1847." That seemed to imply an increase in the flood-water; that there was not so much undergrowth and so on, to hold water like a sponge: but further on he stated "this has the effect of reducing the maxima points of flood supply," so that it was not quite clear whether he thought that the floods were greater or less than they were originally. To sum up, he should be thankful if something could be done towards giving statistics of the nature of the Author's diagram, but statistics of an unexceptionable value.

Mr. Wells. Mr. L. B. Wells said that his experience did not extend to Ireland; he was chiefly interested in the question of sluices, and had seen the Belleek sluices erected in the maker's yard. Acting upon a report by Mr. Leader Williams which was approved by Sir John Hawkshaw, their Consulting Engineer, the Weaver Trustees determined some years ago to improve the navigation by diminishing the number of ponds below Northwich by one-half, thus necessitating the alteration of the water-level in many of them. One of his first duties upon becoming their Engineer in 1877 was to advise the Trustees as to the best method of raising the surface of a pond 4 feet 9 inches, some 6 inches higher than the pond above it. Of late years, the people, in the Weaver Valley especially, were alive to their own interests, and if there were floods endeavoured to get damages from the Trustees. He felt the juncture was a critical one, and that he could not recommend a fixed weir such as existed on the navigation, with small side sluices to let off the water when repairs were necessary. He therefore advised that the water should be penned up entirely by sluices, and placed himself in communication with Mr. Stoney. When Mr. Stoney had spoken during the discussion of the courage of his countryman, Mr. Price, he had forgotten the Englishman who had been quite ready, years before, to adopt his sluices, and in fact recommended them. The question between them was one of cost. He was satisfied that the sluices would work, and that they were thoroughly sound in principle. He was however left to his own resources, and determined that at any rate he would not have surface sliding over surface, but would lessen friction in some way, and to do that he introduced four rollers.

Mr. Wells.

The sluices were made somewhat on the principle of those described. There was a plate girder near the bottom, and another 4 feet from the top. At the back he put four rollers, which travelled on a planed surface of iron attached to the pier. The sheathing of the sluices was of timber. The rollers worked in a recess as in the Belleek sluices, and a stop-water, in principle the same but different in detail, was adopted. He had great difficulty in getting assent to sluices being used in lieu of the old type of weir, and thus breaking through the practice on the Weaver. His report was referred to Sir John Hawkshaw, who confirmed the proposal of having a movable instead of a fixed weir, and eight sluices of 15 feet span and 13 feet deep, about half the size of the sluices at Belleek.• There was a depth of 4 feet of water on the lower side. The lift was 13 feet. This provided for a drainage area of about 530 square miles, and was calculated to run off 0·50 inch of rainfall in twenty-four hours. It had been found sufficient, and the effect in conjunction with the removal of two fixed weirs, and other alterations in the way of deepening and widening the navigation at certain places, had proved most beneficial in the valley. To go back to the year 1850, there was a flood at Northwich of 12 feet on the navigation gauge; in 1863, of 10 feet 2 inches, and in 1872 there were floods of 8 feet and 9 feet 3 inches on the same gauge. In 1879 there was a flood of 7 feet 9 inches, and in 1880 of 7 feet 2 inches. Since the construction of the sluices at Dutton 4 feet had been the maximum flood, and that happened four years ago. It was necessary to be especially careful at Northwich. The town, as was notorious, sank owing to the subjacent rock-salt melting; it had subsided several feet in a few years, and boats were used in the streets during floods. Finding the sluices at Dutton acted so well, he applied them when improving the upper navigation between Northwich and Winsford, wherever he had the opportunity. There were now sixteen places on the Weaver in which the spare sluice provided could be fitted if it should be required. No spare sluice, however, had been needed, and everything had worked satisfactorily. Having eight sluices at Dutton, he constructed a treble geared travelling crab, by which one man could raise a sluice slowly, and this could be done with speed by several men on the other purchases. Where there were fewer sluices, a screw with rack and pinion had been used, and had answered the purpose well. The men in charge were in telephonic communication with each other, and with the Engineer's office, to which telegrams were sent from the head waters of the Weaver and Dane whenever the rainfall was

Mr. Wells. excessive. He quite agreed with what Mr. Barry had said with regard to the large amount of land flooded without any very extraordinary excess of rainfall, and he hoped that when the next cycle of wet weather came the British public would be educated as to the advantage of having movable weirs in the place of short fixed weirs which they now put up with, although grumbling very much when their lands were damaged. By removing obstructions and improving the river channels generally great benefits would ensue; this had been proved in the Weaver valley.

Mr. Tripp. Mr. W. B. TRIPP said that in gauging with current-meters under Mr. Deacon, he had found one practical difficulty in that when a flood was rising all the little streams washed off the ground small matters such as straws and twigs, which tended to interfere with the revolutions of the meter. But immediately a flood had gained its height and began to fall—it seemed never to be quite stationary—the water became comparatively clearer and the difficulty of using a current-meter was then very much decreased.

Mr. Hawkshaw. Mr. J. CLARKE HAWKSHAW could corroborate what Mr. Barry had said with regard to the prevalence of the idea that floods were to some extent beneficial. That idea was held, not only by farmers in Ireland, but by some in England. In 1878, Sir John Hawkshaw was asked by the Thames Valley Drainage Commissioners to report on the best means of dealing with the upper 60 miles of the River Thames; and his instructions were that the works should not be designed to get rid of the floods altogether. They believed that floods to a certain extent were beneficial, and that the works should be designed for the purpose of mitigating them. The great difficulty in designing drainage works for the prevention or mitigation of floods was to settle the amount of rainfall over the drainage area which the new or the improved channels should be calculated to carry away. In some districts that was more difficult than in others. In the Upper Thames it was especially difficult, because of the great variety in the character of the soil. Part of the drainage area consisted of clays, but a great part of very permeable rock, such as oolite and chalk. In the Lough Erne district works were provided to carry away 0·25 inch of rain in twenty-four hours; in the works Sir John Hawkshaw proposed for the Thames Valley Drainage they were designed to carry away 0·12 inch. Of course those works were not designed to get rid of the floods altogether, but only to mitigate them. In some of the tributaries a larger amount was provided for, namely 0·20 and 0·25 inch. The works proposed in the report had not been carried out entirely; but certain parts of the river had been dealt with in the manner suggested; and, so far, it might be said that the amount provided for, 0·12 inch, appeared to be sufficient for the purpose of getting rid of the floods which did most damage. Mr. Hawkshaw.

Mr. W. Berrell desired to supplement the statement of Mr. Hawkshaw with reference to the works of the Thames Valley Drainage. He had mentioned that the provision for 0·12 inch of rainfall in the Thames Valley against 0·24 and 0·25 inch in the Lough Erne district offered a great contrast. The drainage areas were very nearly equal. From what he knew of the district of Lough Erne probably the greater quantity of water to be carried away in the short time was due, not so much to the heavier rainfall, as to the greater steepness of the tributaries rising in the limestone hills surrounding Lough Erne, bringing down water into the lake with great rapidity and causing a very sudden rise in the level of the water in the lake; whereas the same amount of rain falling in the Thames Valley would not raise the level of the water to the same extent. The works Mr. Hawkshaw had referred to had in a great degree been carried out, especially below Oxford; and since 1886, when the greater part of those works were completed, very little of that land had been under water, and that for only a few days, which, he believed, farmers thought did no injury to the crops; they rather liked the land to be warped, as they termed it, to get the slight sediment on the grass, which did not injure it. If that water, however, remained for any length of time, it killed the finer kinds of grass, and the quality of hay became deteriorated. The weirs that had been put up in the Thames Valley drainage-district were not so large as those constructed at Belleek, but they might be called deep-sill weirs; they had, many of them, a depth of 9 feet of water running over the sill below the summer water-level. He noticed that the sluices though 14 feet 6 inches deep, were only raised 9 feet. He presumed that the depth of water running over that sill in flood time was only 9 feet; so that the actual capacity of the sluices built in the Thames per foot of width was equal to the sluices designed for the Lough Erne drainage. Whilst on that subject, he might remark that considerable difficulty had been found, with large sluices designed like those at Belleek, owing to the accumulation of twigs, boughs, and trees, and drift that came down with the surface-water. It would be seen that, as the water rose, the sluices had to be lifted from the bottom, the lip of the sluices being submerged, a large quantity of rubbish accumulated on the upper side, which had to be cleared away. It was a great convenience to have a small upper sluice to lift off just sufficient Mr. Berrell.

r. Berrell. to carry the rubbish through in some part of the weir; but where there was a weir of the full width of the river, with the whole of the water pouring under the sluice, there would be very great inconvenience in times of flood from the amount of rubbish accumulating above it. With reference to the amount of water which passed through the weirs, he should like to examine the gaugings which had been taken, for, from what he could ascertain, the formulas, which had been based on smaller weirs, greatly underrated the amount of water passing through such sluices. The obstruction of the new weirs constructed on the Thames, with frames about 6 inches wide, and spaces of 6 feet between them, was so slight that the water, instead of surging through, seemed to pass through as a steady current, with very high velocity; and it was some long distance below, where the water of high velocity met the more sluggish water, that the broken water occurred.

r. Topley. Mr. W. Topley said that one point of interest was the geological nature of the basin under consideration. The matter was important, because the Paper was so good, even making allowance for the apposite criticisms of Mr. Symons, that it was likely to be taken as a standard reference for some time by engineers; and it was therefore the more important to have precise knowledge of the data submitted. The Paper had referred to the district in question being a limestone area; and comparison was made with other districts, the object of the Author evidently being to take that as a typical limestone area, and to compare it with others not so circumstanced. There were one or two criticisms upon that statement which might be read within the lines of the Paper itself. The Author had stated that "the Erne drainage basin consists almost entirely of calp limestone (except at the north-west where a little gneiss and granite crop up); the rock is fairly absorbent, being a mixture of solid limestone with shaley beds;" and he added, "a glacier deposit of blue boulder-clay overlies the limestone." Everyone knew that, in a question of this kind, if a deposit of clay overlay limestone, it was of very little consequence which rock might lie some feet below. What the engineers had to consider, in a drainage area, was the surface soil, because upon that depended the way in which the rain would run off. He had examined the maps of the Geological Survey, and had found that a very large portion of the district was composed of boulder-clay. The Author had referred to this in speaking of the manner in which the excavations had been made. There was only one place in which rock-cutting was mentioned, namely, Belleek; everywhere else the cuttings were through clay; that referred to low-

Mr. Topley. lying land near the lakes, but it was equally true if applied to the higher area surrounding the lakes, over which the water flowed. The limestone was not entirely limestone. Even disregarding the boulder-clay, it would be a mistake to speak of it as a limestone area in the sense of the centre of Derbyshire, which was a typical limestone area. There was a considerable amount of shale even within the limestone beds; and in the beds above them, forming the course of the watershed, there was a large amount of sandstone. Mr. Symons had referred to the desirability of having a map, showing the actual line of the watershed; and he would suggest that if that were done, it would be desirable to show the nature of the soil, which could easily be done by means of the published Geological Survey maps. He should be happy to supply the information (Plate 6, Fig. 5).

Mr. Price, sen. Mr. JAMES PRICE, senior, in reply to the discussion, on behalf of his son, who was abroad, observed that Mr. Hassard made light of the expenses incurred by the claims of the owners of the Ballyshannon Fishery Company, and said that it was very little more than might have been expected; but it turned out to be ten times as much as had been contemplated; at least, no considerable sum did appear in the estimates of the engineers who had previously considered the question. The Drainage Board paid over £10,000 in compensation and costs for an eel-weir at Belleek, though a better one was constructed in its place, and offered to the owners, who refused to take it as a substituted work; and when the Board fished the weir at a large profit, in order to prove its value, it was restrained from doing so by a decision of the Irish Master of the Rolls, whereby the owners of the fishery got large compensation, and at the same time caught the eels in their weirs lower down; such a result was certainly unexpected. Another reason for the excessive cost was the rate of wages, which rose 25 per cent. from the time of making the estimate till the works commenced; this delay was caused by circumstances not within the control of the Board. In addition to the above causes, the plans had to be prepared from surveys and borings made by other engineers, which showed about one-third of the quantity of rock that had to be excavated, and there was neither time nor money to prepare new borings. Mr. Porter, the promoter, having already spent out of his own pocket many thousands of pounds, and long years having been occupied in the several attempts to accomplish this work, was not prepared to delay the matter further; to have made new borings would have been most tedious and expensive, the bottom of the river being paved, as described, with boulders.

Mr. Price, sen. The position of the sluices, at a distance below the head of the lake, was not determined on with a view to the navigation of these 3 miles of river; the only navigation considered was that in the upper waters; but they were placed at Belleek in consequence of the rapid fall at that place, which took the water away at once without any backwater, so that the sluices were only half as large as they should have been if placed at the head of the lake. If they had been constructed in the position suggested by Mr. Hassard, almost the whole water-power of the pottery-works would have been destroyed, involving enormous compensation, as the wheels gave at present 130 HP., and there was enough water for 200 HP. all the year round. The performances of the dredger had very much improved since Mr. Hassard's inspections; it would now excavate up to 400 cubic yards per day; the smaller output at the earlier stages of the work was due to the surface pavement of boulders. It was always intended, as the specifications would show, that most of the cuttings should be made in the dry by means of stanks, or that the contractor should procure more dredgery plant, as he might prefer. The section suggested by Mr. Hassard had been adopted in two or three places with advantage, and Mr. Price was indebted to him not only for this suggestion, but for his careful measurements and favourable report on the works, and his recommendation to the Board as to a settlement with the contractor for a portion of the works. As Mr. Price, senior, and the Author of the Paper were, in fact, agents to carry out two-thirds of the work, it was necessary that an independent engineer should be employed to measure the work and advise the Board. The remarks of Mr. J. W. Barry were very encouraging, coming from one of the Royal Commissioners on Irish Public Works. However, on the subject of navigation, there was a tendency on his part—shared by others—to underrate the inland navigation in Ireland. No doubt it had not as yet proved of very great consequence; but as Lough Erne, with its unbroken sheet of water 52 miles in length, joined the Shannon navigation from Limerick with that of Lough Neagh and the Lagan terminating in Belfast, it would be a gross error, having regard to what might be in the future, so to lower Lough Erne as to close this great water highway. Besides, a large number of resident proprietors had their demesnes on the margins of the lakes, no doubt attracted there by the beauty of the scenery, and by facilities for yachting, who would never have given their consents to the scheme were the lake to have been lowered even by 2 feet; and it was a proposal to

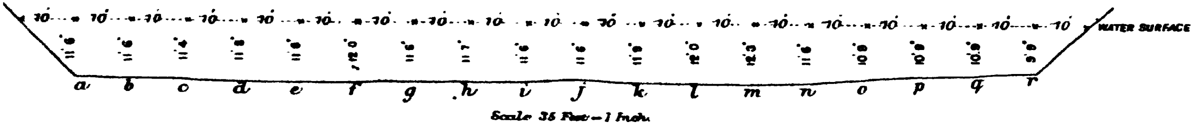
Mr. Price, sen. reduce the summer level of the lower lake by 2 feet, as embodied in almost all the previous schemes, that prevented those projects obtaining the necessary assents of two-thirds of the proprietors. Moreover, were the lakes run down to their very lowest, on account of the quickly-shelving margins, only an insignificant area of land would have been laid dry, which would consist principally of stony shores, and these must have been flooded in all except very dry seasons to gain sufficient head to drive the water through the channels from the lower lake to Belleek, unless these channels were enlarged and thereby the expense of the drainage doubled; so that, on the whole, the maintenance of the navigation—which was used by the country-people—was a matter not only of prudence but almost of necessity. The commercial advantages had been shown to be at least double the annual charge upon the loan, which would itself be paid off in thirty-five years. The Author did not assert that a depth of 60 inches of rainfall was run off every summer from its storage in the ground, but that such a quantity was stored there, and would form a reserve in case of a succession of dry years. As Mr. Vernon-Harcourt had himself resided in Ireland, and seen the sluices, his general approval was much appreciated. The Author stated that "thorough drainage" increased the floods, but arterial drainage did not do so necessarily. On neither of these points could any general rule be laid down, as everything depended on local circumstances. For instance, in the case of a long river with a great many tributaries coming in together, when the catchment area had an expansive head, if all these tributaries were improved simultaneously, the floods would be greatly increased, as the large area of flooded lands, which allowed the waters to escape over them, both mitigated and delayed the floods before the drainage took place. On the other hand, in the case of a river with uniform width of catchment, and tributaries coming in like the branches of a fir-tree, by draining some of the lower tributaries, the floods from them would have passed away generally (not always) before the flood from the upper tributaries could arrive; in this case, arterial drainage diminishes floods, besides the diminution from the cause assigned by the Author. There was hardly any catchment-area where suitable sites for forming artificial lakes could not be selected, as they might be on the worst land adjoining the main river, or any of its tributaries, no matter where, so as it took up a portion of the flood. The criticisms of Mr. Symons were very fair; neither his son nor he could give the time to arrive at the high degree of accuracy in the investigation that he desired.

Mr. Price, sen. They were indebted to him for the returns of rainfall, as both the gauges from which they took the rainfalls were, he believed, under his direction. Still, there was so much hill district unprovided with gauges in the Lough Erne catchment area, that to have taken an average of three lowland and one hill gauge would have given too small a rainfall. After careful consideration they concluded that the mean of the two gauges, at Florence-Court and Colebrook, represented the average rainfall. It was quite natural that the loss from evaporation in Ireland should have its periods of maximum and minimum different from what it was in England; thus, in Ireland the lowland meadows were still uncut and very luxuriant in August, and held the water as a sponge, exposed to evaporation; in fact, the light showers hardly reached the ground. Mr. Symons objected to a flowing line, in Fig. 2, representing the supply from rain, which was not exactly the rainfall; but the flow of yesterday was merged into the flow of to-day, and continued after the rain had ceased. Figs. 3 and 4, which were only intended to represent May and June, 1888, seemed to agree with Fig. 2. The diagrams showed what took place during these years, whatever conclusions might be derived from them, and they certainly represented actual phenomena. As to Mr. Wells' remarks, Mr. Price fully believed that he, as an Englishman, would have had the courage to use Mr. Stoney's sluices, had he the same unfettered choice and opportunity. It was not clear whether he only used wheels to mitigate friction, or suspended live rollers, of which it may be stated that they almost annihilated that force opposed to motion. With reference to Mr. W. Berrell's assumption as to there being only 9 feet depth of water when all the sluices were fully open, he believed there would not be so much as 9 feet; probably during a high flood, with all the sluices open, there would be only 7 or 8 feet in depth passing under the sluices, which would all stand clear of the water. As yet it had not been customary to pass off a full flood as quickly as possible, as the current interfered with the dredging operations above the sluices, so that the exact depth was still undefined. The fact of this small depth of water when passing full floods arose from the free fall below the sluices, which greatly increased the velocity through them. There had been no trouble with rubbish accumulating behind the sluices, as they were opened in succession, no one sluice being left too long shut; and the slightest opening allowed the rubbish to pass. Mr. Topley's remarks on the geological features of the district were of the greatest importance; the district around Lough Erne had a considerable variety of formation. He subjoined a

Table of experiments on measurement of discharge by ascending floats:— Mr. Price, sen.

EXPERIMENTS ON MEASUREMENT OF DISCHARGE BY ASCENDING FLOATS.

Fig. 4.



This section was at a place a short distance (150 yards) above the sluices; the water was disturbed in its natural flow by the proximity of the sluices; it would be most difficult to measure the discharge by any other method than the above. Two sluices were raised 4 feet each. Head of water over centre of orifice = 10 feet.

Position on cross section.	Distance *x y*.	Time of Ascent.	Corrected mean velocity (V.)	Discharge depth × 10 × velocity (V.)	Surface velocity.	Position on cross section.	Distance *x y*.	Time of Ascent.	Corrected mean velocity (V.)	Dischargedepth × 10 × velocity (V.)	Surface velocity.
	Feet.	Seconds.	Feet per minute.	Cubic feet per minute.	Feet per minute.		Feet.	Seconds.	Feet per minute.	Cubic feet per minute.	Feet per minute.
a	22	13½	103	11,850	97	*j*	27	12	145	17,040	120
b	22	11¾	120	13,800	116	*k*	28	12¼	146	17,520	120
c	25	11¼	143	16,210	127	*l*	28	12¾	140	17,150	120
d	25	10½	154	17,960	127	*m*	29	13	143	16,440	113
e	27	11¼	152	18,240	120	*n*	30	14½	131	14,080	104
f	26	11½	146	16,790	120	*o*	29	14¾	124	13,330	85
g	26	11½	146	16,940	127	*p*	22	14¼	111	11,930	78
h	26	11½	146	16,790	120	*q*	18	16	70	7,520	74
i	26	11¼	146	16,790	136	*r*	6	16	23	2,240	53
					Total discharge in cubic feet per minute . .					263,560	

Mr. Price, sen. The corrected velocity was found by deducting one second from the time of ascent (being time occupied in float assuming a uniform velocity), and deducting from $x\,y$ the distance passed through in one second at velocity of bottom stratum, *Fig. 1* (p. 95); a bottom velocity was assumed, if not quite correct, the error was inappreciable, or the bottom velocity could be measured. The surface velocity was less than the mean velocity on account of the draught of the sluices disturbing the regular flow. In order to compare this discharge with what the sluices were discharging at the same time, the following calculations had been made. The apertures through which the water passed at the sluices = 2 (29 × 4) = 232 square feet, one portion of this water might be taken as approaching the sluices with a velocity of 146 feet per minute, and flowing through rectangular apertures, having the bottom continuous with that of the reservoir above it, and each side with smooth curved channels leading up to the sluice; the border under such conditions was 37 feet in length, and to this a coefficient of 0·96 might be applied; the rest of the border being the top (29 feet) would be in the condition of water flowing through an orifice in a thin plate, to this he applied the coefficient of 0·62, therefore the average for the whole aperture was $\frac{37 \times 0{\cdot}96 + 29 \times 0{\cdot}62}{66} = 0{\cdot}81$, 66 = whole border. Thus one portion of the water had a theoretical velocity

$V = 481{\cdot}8\sqrt{H}$ . . . . = 1,523 feet per minute.
Velocity of approach 146

Total velocity 1,669

Which multiplied by section 232 square feet and divided by 2, being half the water with a coefficient 0·81, gave as discharge of water arriving in the line of sluices $\frac{232}{2} \times 1{,}669 \times 0{\cdot}81 = 156{,}819$ cubic feet per minute. The other portion of the water lost its velocity entirely by coming up against the two sluices that were shut, and then it had to pass round the sharp points of the cut-waters, so as to arrive at the sluices; therefore, the theoretical velocity should be multiplied by a coefficient of 0·62, thus:—

Discharge = $\frac{232}{2} \times 1{,}523 \times 0{\cdot}62 =$ 109,534 cubic feet per minute.
Adding formerly calculated discharge 156,819 " "

Total 266,353

Which very nearly agreed with the discharge calculated by the ascending-float method, namely 263,560. This agreement in the results proved that the ascending-float method was accurate, even in disturbed water, and would be probably more so in regularly-flowing water. It also proved that the coefficients were approximately correct. From the above considerations it was manifest that these coefficients varied according to each particular position of the sluices. As either one, two, three, or four might be partially or entirely open, an interesting series of experiments might be made to determine the discharges under all possible conditions. The above observations with the ascending-float were made by Mr. Samuel Armstrong, Clerk of the Works at Belleek, who now had charge of the opening and shutting of the sluices.

Correspondence.

Mr. Coddington. Mr. JOHN GEO. T. CODDINGTON observed that the Author's advice as to making drainage-channels with an increased depth in the centre, accorded with his own practice during the last three seasons in dealing with the gravel shoals of the River Suck. A deep cut was made near the centre of the stream under shelter of dams, and steam-pumps were almost constantly employed during its excavation, at first in removing the surface-water, and afterwards to keep down percolation through the clean gravel which was the original surface of the bed, and which underlay the dams. When this expensive first cut was completed, the summer flow was carried by it at a moderate depth, generally 2½ to 3 feet, and the remainder of the required flood-sectional area was then removed in the dry at a cheap rate. This widening out had a level bed, not hyperbolic, as recommended by the Author, as the latter form of cross section would not be practicable where there were so many shoals in different circumstances of width, depth, and material. The aggregate length of forty-five shoals already pierced amounted to about 12 miles, spread over a distance of 30 miles.

Mr. Crimp. Mr. W. SANTO CRIMP remarked that during the years 1879 to 1881, when engaged upon the construction of sewerage works for the Croydon Rural Sanitary Authority,[1] he gauged the River Wandle and a small tributary, the Graveney Brook, daily. The areas of the watersheds above the site of the sewage works at Merton were 54,000 acres and 5,510 acres respectively; that of the Wandle consisted almost entirely of chalk, whilst that of the Graveney was of London clay, &c. He was fortunate in being able to include in the gaugings the results of the exceptional floods of the 29th of May, 1879. Rain commenced to fall on the 22nd, and there was some to register on every day up to the 28th. The falls at Merton were as followed:—22nd, 0·23 inch; 23rd, 0·11 inch; 24th, 0·27 inch; 25th, 0·02 inch; 26th, 0·21 inch; 27th, 0·27 inch; 28th, 1·39 inch. Up to the 28th, the discharge of the Wandle did not reach 4,000 cubic feet per minute, but averaged 3,650 cubic feet, whilst the average of the Graveney Brook was 99 cubic feet per

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[1] Minutes of Proceedings Inst. C.E., vol. lxxvi. p. 317.

Mr. Crimp. minute. On the 28th, the respective discharges were 4,704 and 456 cubic feet per minute. On the 29th, the maximum discharges were:—Wandle 23,939, Graveney 14,308 cubic feet per minute. The Wandle was always gauged below the junction of the Graveney Brook, so that the volumes relating to the Graveney Brook must be deducted from those of the Wandle in order to ascertain the true discharge of the latter river. On the occasion of the flood, the discharge from the Graveney Brook watershed reached a maximum of $\frac{14{,}308 \text{ c. f. m.}}{5{,}510 \text{ acres}} = 2\cdot6$ cubic feet per minute per acre very nearly, whilst that from the Wandle was $\frac{23{,}939 - 14{,}308 \text{ c. f. m.}}{54{,}000 \text{ acres}} =$ $0\cdot178$ cubic foot per minute per acre. The figures were striking, as showing the immense influence of different geological formations upon flood discharges.

Mr. Manisty. Mr. E. Manisty remarked that the great advantages of Price's single-chain grab over a grab requiring a special crane had come prominently under his notice on the Eastham section of the Manchester Ship Canal. There had been a slip in one of the estuary banks; the slip consisted of puddle clay. It was out of the reach of any ordinary crane, and too deep to wheel out. To remedy this, he took a 5-ton steam-crane, put on a 40-foot wooden jib, and a Price's grab of ¾ cubic yard capacity, and got out the clay in lengths; taking off the grab and putting on a skip was the work of a few minutes, and then the crane filled in with rubble the hole made by the grab. He was thus able to do the work cheaply and quickly. On another occasion he had a Ruston and Proctor steam-navvy working in running sand, and it was almost impossible to get out the "corners." He put a crane and grab on the top, and, by means of a long jib, was able to excavate the "corners," and thus let the navvy continue working. There was always some such special work for the two grabs he had in use, and therefore he had never fairly tested them for quantity, but had seen enough to say that a grab of 1 cubic yard capacity would raise 30 or even 40 cubic yards of material in an hour under favourable circumstances. He had, with a whole-tine grab, excavated clay so hard that the most experienced workmen did not expect it would be touched. One advantage possessed by this grab was that, suppose it caught hold of a pile, or something like that, when deep under water, the headgear was loosed off the crane, and lowered down till the catches passed over the disk on the top of the bucket; as soon as this was done, the chains of the head-gear were made fast to the crane-chain, and the craneheaving opened and brought up the bucket. This was one of the advantages of the pieces at the points of the catches. Mr. Manisty.

25 March, 1890.

SIR JOHN COODE, K.C.M.G., President,
in the Chair.

The discussion on the Paper by Mr. J. Price, jun., on "Lough Erne Drainage," occupied the evening.

1 April, 1890.

Sir JOHN COODE, K.C.M.G., President,
in the Chair.

The following Associate Members have been transferred to the class of

Members.

Edward Monson George.
John Wallis Shores.
Arthur Ross Wilson.

The following Candidates have been admitted as

Students.

Henry Stewart Attfield.
Francis Gibson Baily, B.A.
Lalit Mohan Bose.
James Henry Heap.
Gilbert Reginald Henderson.
Percy Alexander Johnson.
Alexander McIntosh.
Norman Wilson Pictor.
Reginald Todd.
Arthur John Wheatley.
Reginald George Wildish.
George Hearsey Otto Ziegler.

The following Candidates were balloted for and duly elected as

Members.

Archibald Barr, B.Sc.
James Edward Henry Gordon.
Andrew Frank Higgins.
William Jaffrey.
Arthur Pou.
Adolpho José Del Vecchio.

Associate Members.

Percy Vavasseur Appleby, Stud. Inst. C.E.
Charles Armitage.
George Benjamin Blake.
Arthur Richard Brown, Stud. Inst. C.E.
Charles Stanley Clarke.
Robert Cockburn.
John Henry Cordner-James.
Charles Edward Davenport, Stud. Inst. C.E.
Vernon Warburton Delves-Broughton, Stud. Inst. C.E.
Mark Edward Drury, Stud. Inst. C.E.
Walter George Forder.
Alfred Forrest.

Associate Members—continued.

William Fraser.
Ernest Talbot Cope Garland, Stud. Inst. C.E.
Arthur John Lund Grimes, Stud. Inst. C.E.
Oscar Guttmann.
Carl Hall.
Frank Harris, B.A.
Philip Eliot Hodgkin, Stud. Inst. C.E.
John Cartmel Holme.
Mordaunt Leyburn Maclean.
Arthur Edwin Mills.
Stewart Binny Murray.
Owen Ordish.
Frederick Palmer.
William Pollock, Stud. Inst. C.E.
Christian Heinrich Röhe.
William Pole Routh.
Adam Scott.
Herbert Shaw, Stud. Inst. C.E.
William Henry Tomkins.
Charles Hockin Treglown.
George Johnston Webster.
William Fischer Wilkinson, Stud. Inst. C.E.
Harry Atchison Willis, Stud. Inst. C.E.

Associate.

Edward Coward.

(*Paper No. 2455.*)

"The Barry Dock Works, including the Hydraulic Machinery and the Mode of Tipping Coal."

By John Robinson, M. Inst. C.E.

Site.

Barry Dock is situated on the north shore of the Bristol Channel, between Barry Island and the mainland, 7 miles south-west of Cardiff, and 31 miles east of Swansea. The dock occupies the eastern portion of the Channel, which formerly ran between the Island and the mainland, about ¼ mile in width. The western portion of the Channel has been reserved as a harbour for pilot-boats and small coasting craft. The greater portion of the dock area consisted of mud and silt, with some sand and a few thin layers of peat; but the eastern end was composed of rocky marl and magnesian limestone. Deers' antlers, trunks of trees, human remains, fishermen's bone netting-needles, &c., were found in the excavations. The strata are broken up by numerous faults, the principal one running from Warren Tump, across the eastern end of the dock, to the north-west corner of Barry Island, and to Coldknap Point.

HISTORICAL NOTICE.

In 1865 a railway was promoted by an independent company to run from the Great Western Railway at Peterston to Barry, and from Barry to Sully. In the following year an Act was passed for the formation of shipping places and a tidal harbour at Barry. Owing, however, to the commercial crisis of 1866 the promoters were unable to obtain the necessary funds, and these schemes were never carried out.

A Bill for the construction of the dock and railways was submitted to Parliament in 1883, when it passed through the House of Commons, but was thrown out by a Committee of the House of Lords. In August, 1884, however, an Act authorizing the works was obtained. The first sod was cut by Lord Windsor in November, 1884; the work was then carried on uninterruptedly, and the dock was opened for traffic on the 18th of July, 1889.

GENERAL DESCRIPTION.

The works (Plate 7, Fig. 2) comprise a tidal basin of 7 acres, a dock of 73 acres at the top of the slopes, and 62 acres at their toe, and a timber pond of 24 acres. There is also a graving-dock belonging to a private company. The dock, as originally designed, had a water-area of 40 acres; but the foundations for the south wall were set further back, in order to obtain firm ground, and the area thus increased.

The Barry Dock is designed to afford efficient accommodation for the shipment of coal from South Wales, the annual exportation of which, from Cardiff and Penarth alone during the past twenty years, has risen from 3,000,000 to 11,000,000 tons. Barry Island is about 1 mile in length, by ½ mile in width, and rises 120 feet above mean sea-level; and the mainland north of the dock also rises considerably. The site is therefore well sheltered, particularly from south-westerly winds, which, together with easterly winds, are the strongest in the Bristol Channel. The entrance to the dock is at the eastern end of the island under the high land of Nell's Point, which excellent natural shelter is supplemented by two outlying converging breakwaters; these protect the entrance to the south, where the "fetch" is 14 miles, and to the south-east with a fetch of 16 miles. The approach to the breakwaters is free from rocks and shoals, and is also efficiently lighted. Between the heads of the breakwaters there is an opening of 350 feet, which is close to deep water. There is good anchorage ground for the largest vessels to the east of the dock, between Barry and Sully Islands, a distance of 3 miles. The tidal range is 36 feet at ordinary spring-tides, increasing to 40 feet at extraordinary springs; and 19½ feet at ordinary neap-tides, diminishing to 16 feet at extraordinary neaps. The dock is not fed with water from any river. A channel has been dredged between the breakwater heads and the entrance to the dock to a depth of 1 foot below the lowest part of the sills. On the seaward side of the entrance are two timber jetties, 200 feet long, with horizontal fenders along the face (Plate 7, Figs. 7, 8, and 11). The entrance is 80 feet wide, with a single pair of wrought-iron gates, and gives access to a basin 500 feet wide and 600 feet long. Between the basin and the dock is the passage, having also a width of 80 feet. The sides of the entrance and passage have two horizontal elm fenders for the protection of ships. The depth of water on the working part of the sills, which are curved, is as follows:—

High-water ordinary spring-tides.	37·7 feet
Low-water ordinary spring-tides	1·6 „
High-water ordinary neap-tides	29·3 „
Low-water ordinary neap-tides	9·7 „

The available depths, however, at the centre of the inverts are 3 feet more. On the west side of the entrance is a culvert having a minimum width of 10 feet, with a pair of greenheart sluices, raised and lowered by direct-acting hydraulic cylinders and pistons. There are no sluice-ways through the walls of the entrance or passage; but twelve sluices, having an area of 200 square feet, are provided in the gates. In the passage is a pair of wrought-iron gates; and there is a culvert on the west side 8 feet in diameter. The culverts serve to draw down rapidly the water when the flow of water through the gate sluices becomes slow from the decreasing head. The basin is used as a large lock, by lowering its water-level to the level of the water outside; and, as a rule, the dock is worked to the tide of the day, but at low neaps the basin is used wholly as a lock. As the middle of the sills is lower than the bottom of the basin, a channel has been made along the basin, between the entrance and passage, 1 foot deeper than the lowest part of the sills, so that vessels which can pass over the sills can enter the basin. The dock is 3,400 feet in length, the maximum width being 1,100 feet, divided into two arms at the western end by a mole. The northern arm is 1,500 feet long and 500 feet wide, and the southern arm 1,200 feet long and 300 feet wide. The full width is left at the eastern end, for a length of 1,600 feet, where the largest vessels can swing, even when all the berths at the tips and quays are occupied. The bottom of the dock (which is not puddled) is 20 feet below mean sea-level, and the quay roads are 26½ feet above the same level.

Along the northern side of the dock are eleven high-level fixed tips, resting on masonry foundation towers, from 174 to 300 feet from each other, so as to suit the hatchways of different ships. At four places on the north side, provision is made for movable tips, in order that coal may be put into two hatchways of a ship at one time. Where the fixed and movable tips stand, upright walls are built; and between these are slopes, 1¾ to 1, protected by pitching laid on a bed of 15 inches of broken stone. These slopes have been made, not only with a view to economy in construction and working, but also for convenience in coal shipping, as they allow vessels lying at the tips to overlap each other. To get the same number of vessels alongside a wall as can be accommodated at the sloping sides would necessitate an increased length of quay berths. Slopes also impound more water for locking and sluicing operations.

The mole is 1,300 feet long and 200 feet wide, with pitched slopes, 1¾ to 1 on the north side, and 2 to 1 on the south side. On the northern side of the mole are three low-level fixed tips, 300 feet apart; and masonry towers for two others have been built, one on the north side, and one to the eastward of the mole.

An upright wall is built along the south side of the dock, on which two low-level fixed tips have been erected, and two or three movable tips will be added. At the western end a low-level coal-tip is erected, similar to those on the mole, but with a timber substructure. Thus provision has been made for nineteen fixed coal-tips and seven movable tips. At the eastern end of the dock are three timber jetties with movable hydraulic cranes for unloading ballast, timber, pit-props, &c. Movable hydraulic cranes are erected along the quay on the south side, at the eastern end (Appendix VIII).

The foundations of the entrance walls rest on magnesian limestone and soft marl. Across the entrance is a minor fault, compact clayey marl being on the sea side, and rocky marl and limestone on the other side of the fault (Plate 7, Fig. 7). Under the sill, at the basin end of the entrance, is a wall of 3 to 1 fine Portland cement concrete, which has been faced with brickwork in cement mortar, to prevent the passage of water through the beds and fissures of the limestone and marl. Springs of salt water were met with, in which pipes were inserted, and carried to such a height that the water did not overflow. They were then filled with cement grout and, after the cement had set, the projecting portion was cut off. Brickwork, in cement mortar, has been used for the inverts, the upper two rings being blue bricks. Under the inverts, 3 to 1 Portland cement concrete has been laid; elsewhere, 6 to 1. The pointing sills are inverted, and straight on plan, and are granite ashlar, from Cornwall. The hollow quoins and heel-post stones are also granite ashlar. The side walls have no batter, except near the bottom. They are constructed with mountain limestone, from near Wenvoe (about 4½ miles north of the dock), and faced with hard red sandstone, procured from the shore at Sully, much of which was quarried below high-water of spring-tides. The walls surrounding the basin are founded on rock, benched to receive them; they are built of large blocks of mountain limestone, faced with hard red sandstone, squared, but not in courses. They are 50 feet in height, and have a vertical face, but with a curved batter at the base. They are 7 feet thick at the top, and 17 feet where they begin to curve. The cross section of the wall is varied in different places, to suit the nature of the ground upheld by them. Granite coping, 3½ feet wide and 1½ foot thick, with Portland cement dowels in the vertical joints, is laid on the basin and dock walls. The top of the coping is 6¾ feet above high-water of spring-tides. The space between the toe and the excavation in front is filled with 9 to 1 cement concrete. The south dock wall is similar to the basin walls, with the upper portion a little thicker; the top is dressed for 20 feet on the face, and projects 3 inches beyond the lower portion, which is left rough, so that vessels may not rub against the latter when the water falls in the dock. At the eastern end, the foundations are on rock, and elsewhere on marl or shale. The masonry was set by steam derrick-cranes, and also by steam travelling-cranes. Cast-iron 4-inch pipes were placed through the walls; and rubble-drains, 2 feet square, were built at the back. In the mouths of the pipes brass valves were inserted, to relieve the pressure at the back of the walls during construction, and in the basin when the water is lowered. Dry material was used for filling behind the walls; and any mud was cut away in long benched slopes, and dry materials substituted (Plate 7, Figs. 4 and 5, and Plate 9, Fig. 6).

The masonry towers for coal-tips on the north side of the mole are founded on a bed of 7 to 1 cement concrete, 5 feet thick, resting on mud. Previous to commencing these coal-tips, experiments were made to ascertain the bearing power of the mud, by building three brick piers thereon, forming a triangle, which were weighted with rails. The mud bore a weight of 2 tons per square foot, when it suddenly yielded. The foundations of the coal-tips on the mole are therefore given an area over which the distributed weight is only 1 ton per square foot. On the west slope of the dock, a timber coal-tip is built, resting on piles driven into the mud, surrounded with concrete just below the dock bottom. On the north side of the dock, the masonry tower for No. 11 coal-tip rests also on bearing-piles driven through the mud, with raking-piles in front, the hard substratum below being inclined towards the dock. All the heads of the piles are surrounded with Portland cement concrete.

While sinking for the foundations of No. 10 coal-tip, some strong springs of fresh water were met with, and were conveyed from the foundations, through earthenware pipes, into a cast-iron cylinder, sunk at the south-east corner of the masonry tower. This water was pumped into tanks for the locomotives, and into a reservoir for the supply of the new town of Barry during the construction of the works.

The masonry is set in blue-lias lime-mortar, made from stone excavated on the site of the works, the strata being the same as those yielding the well-known lias stone of Aberthaw, a few miles distant (Appendix I).

The dock is connected with the coalfields by a railway, which joins the Rhondda Fawr branch of the Taff Vale Railway near Hafod (Plate 7, Fig. 1). The ruling gradient against the load is 1 in 400 to the dock, and for the return empty wagons 1 in 120. Branch lines connect the railway with the Taff Vale Railway at Treforest, and with the Great Western Railway at Peterston and St. Fagan's. A branch line, chiefly useful for passenger traffic, connects Barry with the Taff Vale Railway near the Penarth Dock station.[1]

Various levels of the chief parts of the works are given in Appendix II, and details of the quays, mooring-posts, and rings in Appendix III.

Since the opening of the dock, a dam has been constructed east

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	Miles.	Chains.
[1] Length of main line to Hafod	18	69
Length of Peterston Branch	1	79
Length of St. Fagan's Branch	1	9
Length of Treforest Branch	1	54
Length of Penarth Branch	3	60
Total length . . .	27	31

of the entrance channel to enclose the foreshore, and also to prevent vessels being affected by the current flowing and ebbing to and from the foreshore inside the eastern breakwater.

At the entrance, 10- and 5-ton, and 5- and 2½-ton double-power hydraulic capstans are provided on each side; similar capstans are fixed on each side of the passage, and also snatch-heads. On the south quay are four hydraulic wagon capstans; and two hydraulic capstans are fixed at each coal-tip. Details relating to the hydraulic engines, pumps, and accumulators are given in Appendix IV.

The construction of a lock to the east of the basin, with deep sills, is contemplated, so that vessels may enter and depart at almost all states of the tide, in connection with which a channel will be dredged, extending to deep water, a little beyond the breakwater heads.

METHOD OF EXECUTING THE WORKS.

The tide had first to be excluded from the site of the dock and quays, upwards of 200 acres in extent, a somewhat difficult proceeding on account of the great tidal range in the Bristol Channel. This was done by putting three dams across the channel between the mainland and Barry Island, the two outer dams including the whole of the site required for the actual dock works. The intermediate dam was the first to be closed; and the only trouble experienced was the sinking of the earth into the mud, necessitating the frequent raising of the embankment tips. The closing of the second dam, at the western end of the dock, was very difficult, owing to its resting on mud varying in depth from a few feet to upwards of 40 feet. The first portions were made at each end, by tipping earth from wagons run out from the mainland and island; but, on reaching the deep mud, owing to the earth sinking into and sliding away with it, a timber pile viaduct was constructed across the gap, on to which the loaded trucks were run and the earth cast out, thus forming the dams in layers. This method was persevered in until, as the two ends approached each other, the tidal current became too rapid. Two unsuccessful attempts were then made by the contractor to close the gap with earth at low-water neap-tide, the intention being to exclude the tide of the day at low neaps, and then to raise the bank each day above the increase in height of the tide. In spite of every effort, the water gradually gained upon the work, and washed the earth inwards, leaving a gap 80 feet wide, through which the speed of the tide was nearly 5 miles an hour. The method, originally proposed by the engineers, of dropping down shutters between walings, securely fastened to the viaduct piles, after the tide had receded, was now resorted to. The shutters were backed up with a large quantity of stone, together with earth, as rapidly as all the available trucks and wagons, which were standing full, could be brought forward. In this manner the tide was excluded from the western portion of the dock works in July, 1885. A 40-inch cast-iron pipe had been laid through the dam, and rested on a timber platform on the mud. A flap on the outside was closed against the rising tide, and opened when it receded, thus allowing the water left inside to be gradually lowered to the level of the sluice, and the excavation above that level to be proceeded with. The remainder of the water below the pipe was removed by pumping.

The eastern dam remaining to be closed had been made partly of permanent, and partly of temporary, masonry, founded on marl and backed up with earth. To close it, piers of masonry were built, leaving four 15 feet openings, through which the tide flowed, until the openings were rapidly closed with planks, backed with concrete made with blue-lias lime and Portland cement, in March, 1886. The planks were afterwards removed, and the concrete was faced with brickwork in cement mortar. To drain away any water accumulating at this end of the dock, three 12-inch pipes, with valves, had been laid through the bottom of the concrete wall, at its lowest part. Excavation was then proceeded with inside this dam, above the level of the three sluices. To drain the water from the excavation for the dock below the level of the temporary sluices, a sump, 20 feet in diameter, was sunk 10 feet below the bottom of the dock at the north-west end. From the sump a heading was driven, 160 yards in length, into the corner of the dock. All the water which accumulated between the intermediate and western dams flowed along channels to the corner of the dock, thence through the heading to the large sump, and was pumped by a Cornish engine along a wooden shoot, over the western dam, into the harbour. The engine had been removed from the Severn Tunnel works, and was capable of lifting 270,000 gallons per hour; but 150,000 gallons was about the average pumping required. To drain the excavation between the eastern and intermediate dam, another sump was sunk near the entrance, in which were placed two T bob pumps, which discharged the water over the eastern dam into the sea.

When the water was excluded from the site of the dock, between the western and central dams, borings were made to ascertain the exact nature of the strata underlying the dock, which had already been found to be disturbed by faults. These indicated the necessity of setting back the south side of the dock, and of altering the position of the entrance.

The excavation for the dock was carried out in various ways, steam-navvies and grabs being extensively employed. The steam-navvies, when in regular work, excavated, in a day of ten hours, 500 cubic yards of marl, loosened with powder, and in softer material, such as stiff mud or clay, 1,000 cubic yards. For excavating mud from trenches in which water lay, steam-grabs proved the most useful; for lifting rock, portable steam-cranes, with iron skips, were employed. All hard materials for the excavation were utilized for embankments and quay roads round the dock; and the mud was deposited at the back of these, and in trenches for making the works watertight. Special side-tipping wagons, designed by the contractor, were used for tipping the mud and silt. The earthwork being well advanced, masonry was commenced in the dock. The lower portions of the basin walls were built in trenches, cut in the marl and magnesian limestone, before the completion of the excavation forming the basin. For transporting the material, there were thirty locomotives, exclusive of those working on the railways north of the dock. When the work was in full progress, about three thousand men were engaged. During the summer and autumn, the work was carried on also at night, temporary electric and Wells' lights proving extremely useful. The dredged material from the entrance channel was carried out several miles to sea, in hopper barges.

A caisson was erected at the sea face of the entrance, inside the temporary stone dam. It fitted against the quoins of the entrance, and enabled the temporary stone dam to be removed before the works were wholly completed. On the 29th of June, 1889, water was first admitted into the basin and dock by opening the sluices in the 10-foot culvert at the entrance, on a rising tide, the outer gates being closed, and the caisson, in position, resting on the keel-blocks against the quoins. The sluices in the culvert at the west end of the dock were also opened. At that tide the basin and dock were covered with 5 feet of water; at the next, with 18 feet; and at the following tide with 23 feet. The water between the entrance gates and caisson was allowed to flow inwards and outwards at each tide, through the sluices in the caisson; but as soon as the height of the tide exceeded the water in the dock, the caisson sluices were closed, and opened again when the tide receded. On the 13th of July, 1889, the caisson was floated with a draught of 18½ feet of water, and was taken into the basin by a tug, and the tide admitted freely to the dock.

BREAKWATERS. (Plate 7, Fig. 6.)

The breakwaters are formed of rubble, excavated from the basin and railway cuttings; and the sea slope is protected by blocks of mountain limestone, of from 4 to 7 tons. The eastern breakwater is 2,600 feet long; the western breakwater, 700 feet long, is connected with the island by a light timber viaduct. The sea slope of the breakwater varies from 3½ to 1 at the toe, to 1 to 1 at the top; and the inner slope is 1½ to 1. The width of the breakwaters, 8 feet above high-water of spring-tides, is 20 feet; and above this is a rough parapet, 12 feet wide and 5 feet high. The great rise of tide renders it necessary that the breakwaters should be substantial works; they are, at the deepest part, 46 feet in height, and 200 feet in width at the base. The rubble hearting was deposited from wagons; and the large blocks were brought in trucks from a quarry 5 miles away, and placed in position, on the eastern breakwater, by a 10-ton Scotch crane, and on the western breakwater, by a crane fixed on a floating barge. The ends of the breakwaters are vertical, and consist of creosoted timber pilework, 32 feet wide, the intervening space being filled with stones roughly packed by hand. There are seven compartments in each breakwater head, with vertical rubbing pieces on the inside, so as to permit the subsidence of the hearting without damage to the framing.

LIGHTHOUSE.

A cast-iron tower has been erected at the head of the west breakwater, 30 feet in height, having a diameter at the base of 7¾ feet, and at the top of 6½ feet, and a spiral staircase inside. In the tower is placed a fourth order dioptric, occulting, white light, visible in clear weather at a distance of 10 miles. It is 40 feet above high-water of spring-tides, occulting in periods of five seconds (three and a half seconds light and one and a half second dark), worked by eight-day clock-work. The light is suitable for both petroleum and gas; petroleum is used, which necessitates the attendance of a keeper, night and morning, to extinguish the lamp. When gas is used, personal attendance will be required only once a week. On the head of the eastern breakwater is a fixed bright light, half the height of the light on the western breakwater.

DOCK GATES. (Plate 8, Figs. 1 to 3.)

The entrance and the passage are each provided with a pair of wrought-iron gates, having a span of 80 feet, and a rise of 20 feet; they are 48 feet 10 inches deep at the mitre-post, and 42 feet 1 inch at the heel-post. Each gate is divided into fifteen watertight compartments, by decks and bulkheads. There are twelve decks, five of which are watertight; the greatest width of the decks is 8 feet, and the length along the centre line of the gate is 50 feet. Below deck No. 5 from the top, are air-chambers, which are almost sufficient to float the gates, leaving only a small weight to be carried by the roller and bottom pivot. Above deck No. 5, the interior of the gate is open at the back. Access to the air-chambers is obtained by a shaft leading from the top of the deck through this portion of the gate. Provision is made for draining the air-chambers at the bottom of the gates, whence the water can be pumped out. In each leaf there are six sluices, having a combined area of 100 square feet, worked by direct-acting hydraulic cylinders and pistons. The heel-posts, sills, and mitre-posts are of greenheart, the heel-posts having a radius of 15 inches. The heel-post rests on a cast-steel hemispherical pivot, with a cupped saddle, sufficient play being allowed to admit of the gate being thrown out slightly from the hollow quoins by relieving gear, to prevent the wearing of the heel-posts and hollow quoins when the gate is opened and closed. Oil-pipes are brought up to the top of the masonry of the gate recess, and a force-pump is attached to supply the oil for lubricating the pivot and relieving gear. The top anchorage consists of a circular collar, with a bush round an octagonal pin, fixed to the top of the heel-posts, sufficient play being allowed between the sides of the pin and the bush to admit of the free adjustment of the gates against the hollow quoins. The strain on the top pivot is taken by two strong wrought-iron ties, anchored to wrought-iron girders built into the masonry at the back. Forged-steel rollers are fitted to the gates, carried in a frame having a bearing under the gates, and supported behind by a spear, terminating with a screw for adjustment. The roller-paths, constructed of wrought-iron plates on edge, with a steel face on the top, have a radius of 43 feet. The rollers are provided with scrapers fitted close down to the roller-paths. Each leaf contains about 186 tons of wrought-iron, 22 tons of cast-iron and steel, and 1,400 cubic feet of timber. Each gate was erected near the position it had to occupy, and each compartment tested with water. The gates are opened and closed by direct-acting hydraulic rams, a system which has proved very successful (Plate 7, Fig. 7; Plate 8, Figs. 6 and 7; and Appendix V).

To prevent the gates from floating, a cast-steel holding-down bracket is fastened at the top of each heel-post. For the protection of the gates from collision, stout chain cables and wrought-iron box booms are placed in front of the masonry. A combined regulating- and escape-valve has been fixed to each of the gate-machines at the entrance, in order to reduce the pressure from 700 lbs. per square inch to any degree between 500 and 100 lbs. The pressure usually employed for the entrance-gate rams is 250 lbs.; and the ordinary time occupied in opening the gates is one minute and a half, and the same in closing. At the passage, the full accumulator-pressure is admitted to the ram-cylinders; and the time occupied in opening is less than one minute, and the same in closing.

Caisson. (Plate 8, Figs. 4 and 5.)

The caisson, made of wrought-iron and steel, is designed to suit its different positions, at the sea side of the entrance, basin side of the entrance, basin side of the passage, dock side of the passage, future lock, and future graving-dock. It has a length of 85 feet, and a beam, amidships, of 24 feet. The height, at the ends, is 42 feet, and at the centre 48¾ feet. The keel is formed to a radius of 137 feet, to correspond with the inverts; and the meeting faces are lined with greenheart, 15 inches by 7 inches. The caisson is ship-shaped on one side, and flat on the other, so as to enable it to fit against any of the faces now or hereafter to be made, and to be placed, when out of use, in the recess provided for it west of the entrance, where it will not project beyond the line of the wall. It is divided vertically by two watertight bulk-heads, and horizontally by eight decks, two of which are watertight, forming air and water chambers. The lower parts of the skin are of wrought-iron, and the upper 19 feet of steel. Immediately below the bottom deck, which is watertight, there are two sluices, each 4 feet by 2 feet, for lowering or raising the water-level on the two sides. Cast-iron kentledge is packed as ballast in the bottom of the caisson. The caisson contains about 328 tons of wrought-iron and steel, 22 tons of cast-iron, 354 tons of iron ballast, and 335 cubic feet of timber.

GRAVING-DOCK. (Plate 7, Fig. 2.)

The graving-dock is entered directly from the large dock, at the north-east corner, and has an entrance width of 60 feet, and a depth of water on the sill of 24 feet at high-water of spring-tides. The length of the graving-dock, from the entrance sill to the extreme end, is 724 feet, and is divided unequally, by a middle sill, into lengths of 372½ feet and 351½ feet, with a width of passage between the two portions of 58½ feet. At floor-level, the dock has a width of 100 feet, and, at the top, of 113⅓ feet. It has masonry side-walls, with granite altars and coping, and a cement-concrete floor. An open drain runs round the sides, and another one covered with stone slabs down the centre. The masonry of the walls and floor was not made of strength to resist outside water-pressure; but the walls and floor are provided with pipes and escape-valves to prevent pressure. The quantity of water finding its way into the dock is insignificant. Wrought-iron box caissons close both portions of the graving-dock. Four vessels, or even more according to size, may be accommodated at the same time; while by combining the two divisions, a ship of extraordinary length can be admitted. The graving-dock has a double line of keel-blocks. Two 40-inch centrifugal pumps empty either or both divisions of the dock, and are fixed in an engine-house 11 feet below high-water of spring-tides. The pumps can raise 62,000 tons of water in from three and three-quarters to four hours, and may be used separately or jointly as required. An hour and a half is occupied in emptying the inner dock at high-water of spring-tides when no vessels are in, and two hours and a quarter in emptying the outer dock. When vessels requiring their sides to be scraped and washed are in the dock, the water is pumped out gradually, so that the men may do the work as it subsides. When cleaning is not required, the docking is rapid; for instance, the surveyors were able to examine the bottom of a steamer of 1,198 tons in one hour and twenty-five minutes from commencing to haul it into the dock. Ships requiring heavy repairs are placed in the inner dock, and for light repairs in the outer dock. For driving the machinery in the workshops, three gas-engines are employed.

HYDRAULIC COAL-TIPS. (Plate 9.)

High-Level Tips (Plate 9, Figs. 5, 6 and 7).—Trains of from fifty to seventy wagons, mostly containing 10 tons each, are brought down from the collieries. On the high-level, the trains are backed into curved sidings, with gradients of 1 in 233, intended for full wagons. In South Wales, the coal wagons are made with a door at one end only; and coal is not tipped through the bottom of the wagons. The loaded wagons are sent from the collieries with the doors up-hill, to prevent their opening on steep inclines, and, therefore, in most of the docks in South Wales, are turned for tipping. By the plan adopted at Barry, the wagons, when shunted into the sidings, have the doors at their right end for tipping. In case, however, a wagon is sent the wrong way, a table is provided in front of the weigh-bridge for turning it. The wagons are drawn forward, and are run one by one on to the weigh-bridge, and afterwards drawn on to the coal-tips by two hydraulic capstans. Masonry towers, 30 feet square, carry wrought-iron framing, with wrought-iron guides for the cradle. The cradle, which is moved up and down by direct-acting hydraulic rams, has a tip-up table, hinged in front, and worked by a hydraulic cylinder, hung on trunnions to girders underneath the cradle, which travel with it, and capable of tipping loads up to 19 tons. Four cast-iron rams attached to the underside of the cradle, for lifting and lowering, are 8¾ inches, 9⅜ inches, 7½ inches, and 8 inches in diameter respectively, and represent pressures of 18½, 21½, 13½ and 15½ tons on their respective areas. Waste of water is prevented by its being exhausted into the pressure, or into the return-mains. These descending loads (much of the coal from the high-level sidings having to be lowered for shipment) pump water back into the accumulators. The cradle has a maximum lift of 37 feet; and the tip can be used for balancing or bunkering by valves, which are worked by a man stationed in a cabin, containing the lever handles, at an elevation commanding a view of the deck of the vessel which is being loaded. The tips have coal-shoots, anti-breakage cranes, boxes, &c. A chain, fixed to the cradle, is hooked on to the tail end of the wagon, standing on the platform or cradle; and the catch, securing the hanging door at the other end of the wagon, is struck away. The cradle is lowered or raised, and tilted up, discharging the coal into an inclined iron shoot, extending over the ship's hatchway. At the commencement of shipping, an anti-breakage box is suspended at the end of the shoot from a hydraulic lifting and turning jib-crane, fixed near the top of the iron framing, at one corner next the dock. The coal drops into this box, which is lowered into the hold of the vessel, and emptied by the withdrawal of a bolt from a hinged flap at the bottom. To the bolt releasing the flap-door is fastened a chain fixed on deck, thus making the discharge of the box self-acting. By means of the box, a cone of coal is formed as high as the point of the shoot, so as to avoid breaking the coal by dropping from the shoot into the hold of the vessel. The anti-breakage box is then swung out of the way, and the coal allowed to slide from the shoot on to the cone, and into the hold, where it is trimmed. The shoot is tapered towards the point, to check the sliding of the coal, and has a single door across, near the end, for regulating the same; and the butt end is pivoted under to give the shoot a somewhat greater range to the hatchway. For raising and lowering the shoot, and adjusting its level and inclination to suit the varying height of the deck, chains, pulleys, racks and p of ve provided; and the shoot is also secured by safety chains. iode of bottom plate of each shoot are two iron-bar screens for sepable tip when required, the small coal which falls on to the vessel's decw level is removed by an iron skip attached to the anti-breakage The skip is raised and swung round at the same time, and most turned over into an iron spout, down which the screenings pped into a wagon standing underneath the high-level viaduct, and ising of the cradle. On the underside of the shoots, immediately b the screens, are hinged doors to allow of single screening, do screening, or the coal to pass unscreened. As much as 400 have been shipped in one hour at one of the high-level tips, in ordinary course of working. At all the high-level tips, vesselng-any tonnage can be bunkered when light. After the coal has b of tipped, and the cradle brought to its proper level, the empty wagel- is pushed off the cradle, and runs down a road having an inclination of 1 in 80, on to a second weigh-bridge, where the tare is obtained. It is then pushed from this weigh-bridge, and runs down the siding used by empty wagons, which has a gradient of 1 in 70, on to a piece of level road. Here the empty wagons accumulate, and are drawn by the shunting-engine out of the tip siding, up a gradient of 1 in 45, and taken away to the sorting sidings. A difference of level at the crossing rails, of 4½ inches between the lines for full and empty wagons, is met by a sliding rail which is closed by the wheels of the full wagon when passing, and opened again by a weighted crank-lever to allow the empty wagon to pass. Some of the tip sidings have two lines of way which, besides providing additional storage for coal of one quality, are also intended for wagons filled with coal from different pits, so that ships may be loaded with coal mixed in any proportion that may be desired.

Low-level Tips (Plate 9, Fig. 1).—The tips, or hoists, on the mole and on the south side of the dock, differ from those on the north side, inasmuch as the loaded wagons are received into the hoists at the level of the quay, and the empty wagons are returned from the tip at the same level. The wagon has thus to be lifted the required height for tipping. When the wagons are brought from the collieries to the low-level, they are backed into sidings having first a rising gradient of 1 in 120 and then a falling gradient of 1 in 200, where the engine leaves them. They are afterwards pulled forward, down a gradient of 1 in 140, by a hydraulic capstan, on to a weigh-bridge; then on to a turn-table, whence they run on to the tipping-cradle. After being tipped, the wagons are run back on to another agoi-able, where they are turned and pushed on to another weigh-weighe; from this they are pushed off, and run down a gradient of hydra 0, on to a level piece of road, to be drawn away up a gradient wrougin 45 by the shunting-engine to the sorting sidings. The The ctables are so arranged as to avoid any fouling of wagons rams g to and leaving the tip. Each hoist has a substantial wrought-hydr frame, 57 feet high, which supports the shoot for conveying cradl coal into the hold of the vessel, and guides the lifting-cradle 19 t ng its ascent and descent. On one side of the upper part of crad frame is a cabin, for the man who manipulates the valves. The and it lifts wagons, of a gross weight of 19 tons, a clear height of of 1 feet, and tilts them to an angle of 45°. The lifting of the and loaded wagon is effected by two vertical direct-acting of $11\frac{1}{4}$ and $6\frac{1}{8}$ inches diameter, with 37 feet stroke. In the cradle, both these rams are exposed to the full hydraulic pressure; but, in lowering, the larger one is allowed to exhaust freely, while the smaller one returns the water to the accumulator under the influence of the descending weight of the cradle and empty wagon—a contrivance introduced some years ago by Sir W. G. Armstrong and Co. Each hoist is fitted with an anti-breakage shoot, in which the coal, in leaving the wagon, has its motion arrested by a sliding door across the shoot. As the coal slides from the wagon, this door is eased away down the shoot, until, when near the bottom, it opens a passage through which the coal can drop gently into the hold, thus leaving the shoot at a low speed. The sliding door is moved up and down the shoot by a hydraulic cylinder on the side, and is easily controlled by the man in the cabin, who works the hoist. When coaling into small bunker hatches, the shoot is prolonged by hanging a short tapered piece on to the end to contract its wide mouth. The hoist is fitted with a $4\frac{1}{2}$-ton hydraulic crane, carrying an anti-breakage coal-box, holding 4 tons. When commencing to load a ship, the box is filled with coal from the shoot, and lowered into the hold. A cone is thus formed, which both preserves the bottom of the ship from damage, and lessens the breakage of the coal. A hydraulic turn-over capstan for each hoist is provided for moving the coal-wagons, and working the turn-tables. With the exception of No. 12 tip, at the west end of the dock, all the wagons on the low-level lines require to be turned to get them into the hoist, a system necessitated by the limited space available; but at No. 12 tip, wagons run straight on to the tipping-cradle, as on the high-level tips. A movable tip, between tips Nos. 7 and 8, is in course of construction, for coal brought on to the high level; the mode of working to be adopted is given in Appendix VI. A movable tip is also being constructed for coal brought on to the low level (Appendix VI).

In all arrangements for tipping South Wales coal, it is most desirable to prevent rapid motion of the coal, or its being dropped from a height, as this description of coal is easily broken, causing considerable loss in its value.

ROLLING-BRIDGE.

The passage between the dock and basin is crossed by a rolling-bridge, worked by hydraulic power, carrying a single line of railway, and forming also a roadway. The bridge has two steel-plate main girders, with cross girders, and is counterpoised with kentledge at the tail end. The girders are 154⅔ feet in length, 9½ feet in depth, with a flange width of 2 feet, and they span an opening of 83 feet between the bearings. The bridge is carried on 2½-foot steel rollers, a pair close together, bicycle fashion, being fitted under each main girder; and they travel on level steel roller-paths, borne by girders. Two 6-foot trailing-wheels, at the tail end, run on bridge rails outside the main girders, and carry a weight just sufficient to overbalance the front part of the bridge. On one side of the bridge, the treads of the wheels are grooved; those of the rolling-carriage fit a projection on the roller-path, and that of the trailing-wheel fits the bridge rail. On the opposite side the wheels have flat treads. The bridge is moved by a three-cylinder hydraulic engine, placed underneath. When it is started forward, the front end stands slightly elevated, the rails of the trailing-wheels being at a slightly lower level for this purpose; but after the bridge has travelled forward 25½ feet, the trailing-wheels ascend a gradient of 1 in 60 for 75 feet. This lowers the front part of the bridge, and brings it level. The rolling-carriage is now over a table, under which is a large hydraulic cylinder, which pushes out two pairs of rollers, on each side, up inclined planes, and by this means the bridge is lowered or raised. There are two hydraulic buffers at each end to prevent the bridge overrunning in either direction.

ELECTRIC LIGHT.

The dock, sidings, and buildings are lighted by electric light. The general illumination is effected by forty-one arc-lamps, of 3,000 candle-power nominal, each placed on poles, 40 feet above the level of the quay. There are two separate circuits on the north side, and two on the south side, the lamps being alternately arranged in each circuit, twenty and twenty-one lights each, so that, in the event of any accident, it is possible to use every alternate lamp throughout for lighting the dock. It has the further advantage that, when a reduced number of lights are sufficient, it is possible to throw every alternate light, over the whole dock, on to one machine. Glow-lamps light the interiors of the engine-houses, offices, coal-tips, weigh-houses, &c. It has been found necessary, however, on account of the wide distribution of the groups of these lights, and their distances from the engine-house, to use the alternating system, so that the energy may be carried in the mains at the pressure of 2,000 volts, and be transformed, on entering a building, or arriving at a place where the light is required, to the safe pressure of 110 volts. Two identical, vertical, compound engines, of 90 HP. each, with a steam-pressure of 80 lbs. per square inch, drive the dynamos placed between them, by belts direct from the fly-wheels. Two continuous-current machines for arc lighting, to give 1,350 volts, with 15 amperes current, are driven by one engine; and one alternator, to give 2,000 volts with 25 amperes current, is driven by the other engine. Either engine can, if necessary, drive any two dynamos, thus preventing a total breakdown to the lighting by either system.

The Engineers of the dock works were, Mr. John Wolfe Barry, Mr. T. Forster Brown, and Mr. H. M. Brunel, MM. Inst. C.E., the Author being the Resident Engineer.

Details of the quantities of the works are given in Appendix VII.

The Paper is accompanied by a number of drawings, tracings, and photographs, from which Plates 7, 8, and 9 have been prepared.

[APPENDIXES.

APPENDIXES.

I. MORTAR, CEMENT, AND BRICKS.

Previous to commencing the masonry, experiments were made in mortar-making, extending over several months, which showed the great danger in using improperly burnt and slacked lime. The lime, after being slacked, was kept in a shed for seven to fourteen days, and all hard pieces were removed before being ground in the mortar pans. Blue lias stone in this district weighs 168 lbs. per cubic foot, and, after burning, 100 lbs. The proportions adopted were 1 of lime, 1½ of sand from the sea-shore, and ½ of burnt ballast or ashes. Fresh water was used for slacking and mortar-making.

The analysis of an average sample of the cement used was:—

Silica	24·852
Alumina and Oxide of Iron	12·630
Lime	59·132
Magnesia	0·954
Sulphuric acid	1·590
Carbonic acid	0·470
Organic matter, &c., and loss	0·372
	100·000

Bricks used in inverts, arches, and tunnels were made from clay dug on Barry Island, dried in a shed, with flues under the floor, and burnt in Scotch kilns with projecting firing-places on the sides.

II. DOCK-LEVELS AND AREAS.

The following Table gives the levels of the more important parts of the dock above a datum line 50 feet below Ordnance datum:—

	Feet.
High-water ordinary spring-tides	69·70
Low-water ordinary spring-tides	33·60
High-water ordinary neap-tides	61·30
Low-water ordinary neap-tides	41·70
Mean sea-level at Barry	51·60
Entrance channel	28·00
Working height, sill of entrance	32·00
Working height, sill of passage	32·00
Centre part of sills	29·00
Sill of sluice in western dam	44·00
Bottom of basin	30·00
Bottom of channel in basin between entrance and passage	28·00
Bottom of dock	30·00
Bottom of timber pond	56·00
Sill of graving-dock at north-east corner of dock	45·00
Coping of basin and dock	76·50
Top of breakwater heads	78·50
Roads on tip on north side of dock	101·00
Depth of water in dock at high-water ordinary spring-tides	39·70
Depth of water in dock at high-water ordinary neap-tides	31·30

The areas of the dock and works are:—

	Acres.
Water surface	107
Quay roads and lands adjacent	242
Land covered by tide	208
	557

III. BUOYS, MOORING-POSTS AND RINGS.

Thirty wrought-iron buoys were placed in the dock, two in the basin, and two in the entrance channel. Holes were excavated below the bottom of the dock, 13 feet square and 7½ feet in depth, which were filled with 7 to 1 Portland cement concrete, and four cast-iron washers 2 feet square were embedded therein. Two bent 3-inch anchor-bolts were fixed to these, linked together at the top, to which is shackled a stud-link mooring-chain with swivels 45 feet long. The other end is attached to the circular buoys, some of which are 11 feet in diameter and 5 feet 10 inches in depth, whilst others are 10 feet in diameter and 7 feet in depth. They have four watertight compartments with manholes and covers, and are surrounded with a fender of elm timber 1½ foot by ½ foot.

Cast-iron mooring-posts, embedded in cement concrete, are fixed at intervals all round the dock and basin. There are also mooring-rings along the quay wall, and at each of the coal-tips.

IV. HYDRAULIC ENGINES, PUMPS, AND ACCUMULATORS.

The hydraulic power is obtained by two pairs of compound horizontal condensing pumping-engines. These engines are of the tandem type, the cylinders being 16 inches and 28 inches in diameter, with a piston speed of 280 feet per minute. Both high- and low-pressure cylinders are steam-jacketed. The pressure-pumps are of Tannett, Walker and Co.'s differential ram principle, the diameter of the rams being 4⅜ and 6⅜ inches. The boiler-pressure is 80 lbs. per square inch; and at this pressure each pair of engines should be capable of delivering from the pumps into the accumulator 300 gallons of water per minute, at a minimum pressure of 700 lbs. per square inch, when running at the rate of 50 revolutions per minute. The crank-shaft of the engine is of forged steel, 8 inches in diameter. The pump-rams are of brass; and the engines are provided with surface-condensers, together with the necessary air- and circulating-pumps. Arrangements are provided for permitting either pair of engines to discharge exhaust steam into the atmosphere. Condensing water is obtained from and returned to the dock, through 15-inch pipes, by the circulating-pumps. The fresh water delivered into the mains is obtained from a well on the company's property. There are five accumulators in various positions. The diameter of the ram is 22 inches, having a thickness of metal of 2½ inches and a stroke of 26 feet. The four Lancashire boilers are 28 feet long by 7 feet in diameter, and have two flues, each 2¾ feet in diameter. They are designed to evaporate 10½ lbs. of water for every lb. of Welsh coal consumed in the furnaces, and 8½ lbs. for small coal. The hourly consumption of steam per indicated HP. in the steam cylinders of the engine was guaranteed not to exceed 20 lbs., and the hourly consumption of best Welsh coal not to exceed 440 lbs., or 2½ lbs. per indicated HP. The engines, six boilers, three accumulators, and electric-light machinery are contained in a house erected on the south side of the dock. All round the dock are laid 7-inch pressure- and 8-inch return-mains, which are carried across the entrance and passage in grooves. From these mains there are branches to the different coal-tips, 3-inch pressure and 5-inch return. On the pressure-mains are expansion-joints and momentum-valves and stop-valves at convenient intervals; on the return-mains are relief-valves with springs and stop-valves.

Another hydraulic engine-house is being constructed at the north-west end of the dock, to contain engines, pumps, &c., of the same capacity as those in the existing engine-house.

V. GATE-MACHINERY.

In the case of the entrance gate-machinery, the water is admitted to a direct-acting hydraulic cylinder, having a piston 2 feet 5¾ inches in diameter, and a 1¾-foot ram, with a stroke of 25¾ feet, attached to the gate. The cast-steel cylinder is made in three parts, and tested to withstand a pressure of 3,000 lbs. on the square inch. The ram is of iron, cast vertically in one length, and is firmly guided and fixed to the gate by a forged steel cross-head and coupling-pin. Trunnions are cast on the cylinders, projecting above and below, and pivot in saddle-bearings, which also have trunnions on their sides, and these pivot in bearings fixed to the side walls of the ram chamber, thus permitting the cylinder to oscillate horizontally and vertically. The hydraulic rams are of sufficient strength to resist shocks of waves, and to hold the gates rigidly during movement. In the passage gates, the ram is attached to a radius arm near the top, with a slide against the back of the gate, the other end of the arm working on a pin in the holding-down bracket over the top of the heel-post. As the power required is less, the diameter of the piston is only 2 feet 1½ inch, and that of the ram 1½ foot, with a stroke of 16 feet 7 inches. The cylinder for the passage gate-machine is of iron, cast in one length, and tested to a pressure of 2,400 lbs. per square inch.

VI. MOVABLE TIPS—METHOD OF WORKING PROPOSED.

High Level (Plate 9, Figs. 3, 4, 5).—The full wagon runs on the line marked "Full," past the weigh-bridge and turn-table, on to the approach A to the bridge B, at the front end of which is the tipping-table E. The wagon is then tipped by means of the cylinder F into the shoot G, in the floor of which are two screens fitted with doors. These doors can be opened or closed, so as to obtain double or single screened coal; or, both screens being closed, the coal is delivered unscreened. The shoot is closed at the front end by a door H, worked by the cylinder K by means of chains. When the coal-box I is in position for filling, a grid L is drawn forward into the box by the small cylinder O, forming a continuation of the shoot-floor down to the bottom of the box. The grid consists of a series of 4¾- by 3-inch joists, parallel to one another with 3-inch spaces between. The door H is then released, and the coal slides down the grid into the box, the smaller pieces falling between the spaces and forming a bed of small coal in the bottom of the box, the larger pieces resting on the grid. The grid is then withdrawn by the cylinder N, and the large coal falls down into the box and fills it. The box is then hoisted high enough to clear the bulwarks of a ship by the cylinder P; and pressure being released from the jibbing cylinder Q, the jib falls outwards until the box is directly over the hatchway. The chains and pulleys for hoisting and jibbing are so arranged that the box is lowered 2 feet in running from its extreme position in to its extreme position out. The box is next lowered to within 2 or 3 feet of the bottom of the hold; and then pressure is introduced into the cylinder S, which is connected to the box by a 1½-inch chain, while the hoisting cylinder P is connected with the box-doors J, tending to keep them closed. When pressure is introduced into the cylinder S it lifts the box, while the coal, resting to a great extent on the doors, keeps them in their original position; the box, on being lifted, throws them open and discharges the coal. The box is then hoisted clear of the hatchway, and the jibbing cylinder brings the box back to its original position, and it is lowered on to a platform in readiness for another wagon load.

To provide for the various distances between the hatchways of ships, this tip is used in connection with a fixed tip. The ship is brought with one of its hatchways in front of the fixed tip, and the movable tip is then brought opposite one of the other hatchways by the following means. The tip runs on rails along the quay, about 50 feet each side of the centre of the approach: a connecting bridge is pivoted to the tip at U, and the back end runs on a pair of rails on an approach, curved in such a way that the tangent to the rails, at the point where the rollers on the bridge rest, is at right-angles to the centre line of the bridge in whatever position it may be. When the tip is moved, the bridge is drawn away from the approach, making an angle with it. The ends of the rails forming the line of way are made telescopic to allow of small variations in the position of the bridge. Greater variations are made up by loose sections of rails at G, which rest on girders pivoted on the bridge, and sliding on paths under the approach platform. The tip is moved by a hydraulic engine geared to the travelling wheels. The empty wagons are then run back from the tipping-table, past a pair of self-acting points at D, on to the line marked "Empty."

Water for the hydraulic power is taken from the main by a supply-pipe, through oscillating pipes at C, and along the bridge, through a swivel at U, to the valve cabin. The return-pipes run alongside the pressure, through the swivel and oscillating-pipes, which are made double for this purpose. The man in the cabin manipulates all the levers, except that for the tipping-valve, which is worked by the man who attends to the capstan for hauling the wagons on and off the table. The screenings fall into the hopper M, and thence into a wagon placed underneath to receive them.

Low-Level (Plate 9, Figs. 1 and 2).—The tipping-table for a movable tip for coal brought on to the low level has a lift of 30 feet, and is hoisted by four cylinders hung up in the framing of the tip. Two of these cylinders are of sufficient area to balance the moving parts, the other two lifting the load, thus saving water under pressure. The tipping is done by a horizontal cylinder, chains, and wire rope. The cylinder is placed on the top of the tip framing, and the chain passes round the pulleys at the head of the ram and back of the cylinder, and is fastened to a chain barrel keyed to a shaft, which is also fixed to the top of the tip framing. This shaft is keyed to two rope-drums, on each side of which a wire rope is attached, running over pulleys fixed to the cradle frame, and is secured to the bottom of the tip frame to allow of tipping being done at any point within the stroke of the lifting rams. The shoot is raised by two cylinders placed on the top of the tip framing, one for the heel of the shoot and the other for the nose. The shoot is fitted with girders, pawls, and racks, and is so arranged that it will swing on one side should the rigging of a ship come in contact with it. On the top of the tip, a 6-ton crane is placed, which is run forwards and backwards by a hydraulic cylinder, and is withdrawn when a ship is being berthed, so as to be out of the way of the rigging. The hoisting is done by a hydraulic cylinder fixed at the back of the tip frame, to which is attached a small cylinder for releasing the coals from the anti-breakage box. This tip is also provided with an ordinary 2-ton anti-breakage crane.

VII. QUANTITIES.

The quantities of the chief items of work comprised in the execution of the dock are:—

Excavation	5,000,000 cubic yards
Rubble masonry	200,000 ,, ,,
Brickwork	10,000 ,, ,,
Ashlar, chiefly granite	110,000 cubic feet
Timber work	220,000 ,, ,,

VIII. TABLE OF THE MOVABLE HYDRAULIC PEDESTAL-CRANES, LIFTING THROUGH A CLEAR HEIGHT OF FROM 50 TO 56 FEET.

Description.	30 cwt.	2 tons.	30 cwt.	4 tons.
Radius	25 ft. min. 40 ft. max.	29 feet	40 feet	29 feet
Diameter of lifting-cylinders . . .	7½ inches	11 inches	7¾ inches	11 inches small ram 15½ ,, large ,,
Diameter of turning-cylinder . . .	6½ inches	5¾ inches	7½ inches	8 inches
Stroke	10 feet	6 ft. 3 ins.	8 ft. 10 ins.	6 feet 3 inches
Multiplying power.	6 to 1	8 to 1	6 to 1	8 to 1

[DISCUSSION.

Discussion.

Sir J. Coode. Sir JOHN COODE, K.C.M.G., President, said that whatever might be said as to points of detail, the engineers had utilized, in an admirable manner, the peculiar configuration of the site with which they had to deal.

Mr. Robinson. Mr. JOHN ROBINSON wished to explain the term "bunkering" used in connection with high-level tips. It meant that the coal was lifted. The term "balancing" was used when the coal was lowered with the cradle. On all high-level tips there were four rams of different areas, with a view to economize pressure-water. Different combinations of rams could be arranged for lifting a light or a heavy empty wagon, or a light or a heavy full wagon. In the illustration of the gate ram there were some rods and levers for automatically cutting off the stroke of the ram. Those had been disconnected since, and the ram seemed to work well without them. In the basin, the water rose and fell so much that pipes were laid through the walls for relieving the pressure at the back. In the pipes there were brass valves, and when the water fell suddenly in the basin all the valves worked so that there was no danger from the water-pressure behind the walls.

Mr. Brunel. Mr. H. M. BRUNEL said that the positions of the works of the basin entrance and passage were settled largely by the position of the fault shown by the dotted line on the plan, the passage being placed 60 or 70 feet to the south of the fault. The foundations there were on hard rock. Immediately to the north of the passage the bottom on the north side of the fault was very soft. There was a great depth of soft material along the line of the mole eastward. The south wall of the dock was not situated on this soft stuff, but on lias, the stratification on the north side of the dotted line being thrown down. The dock was originally intended to be much narrower than was shown on the plan. When the pits and borings were made after the water was excluded, or about that time, it was thought desirable to move the south wall of the dock to the southward to get it on good foundation, thus making the dock wider, and the mole was added running along the middle, so as to get a greater quayage length. At the entrance there was a small fault, the sill of the entrance being upon hardish marl, though not such hard rock as in the excavation of the basin and of the passage between the basin and the dock. This softer material was watertight, and the whole of the sill of the entrance was upon it; but part of the gate floor was upon

Mr. Brunel. the hard rock. The gate floor was inverted as shown in the drawing to resist any upward pressure coming through the rock in the event of the caisson being ever placed at the north end of the entrance. The basin was really used as a lock at neap-tides, when the water was kept up in the dock, and with the large amount of sluices provided in the gates the dock people ran the water in and out very rapidly; indeed, they thought very little of running the basin down and bringing in a single ship if necessary. The tips along the north side of the dock were arranged at irregular intervals, a pair being placed 174 feet apart, and two other pairs 200 feet apart. The Cardiff people put tips at that distance, and ships were built to suit them so as to be loaded at two places at once. The hinges of the gates were designed to carry the weights of the gates even if scuttled; but it was thought advisable also to have rollers. The graving-dock was very cheap. For economy, the walls of the graving dock were designed to resist the pressure of the ground, but not water-pressure from behind. There were pipes and escape-valves in the walls, but the quantity of water coming in was insignificant, showing that the course adopted had been wise. He did not know whether the advantage of the curved railway lines leading to the tips had been made clear. The trains coming down with the wagons were backed straight into the tip sidings by the engine, and the shunting was very little, which conduced a great deal to the easy and economical working of the dock. All the tips were high enough to lift the wagons and to tip them into the bunkers of the highest ships, so that it was not necessary to pick out a particular tip for a particular ship. The apertures into the bunkers of the ships were, as a rule, higher than the ordinary hatchways, and that led to the expression "bunkering," in the description of the tipping processes, which indicated that a wagon was being lifted higher than usual.

Mr. Walker. Mr. B. Walker remarked that the simplicity about the direct-acting contrivance commended itself to the mind of any ordinary mechanical engineer. When the scheme was before Parliament it was considered one of the greatest difficulties how the gates should be kept steady and in a reasonable position under the varying influences of the flood-tides. Those who had seen the gates knew that the flood had nothing to do with the gates; the workman could put them just where he liked, hold them as long as he pleased, and do what he liked with them. That part of the engineers' intention had been admirably and fully carried out. It was well known that the dock was erected mainly to ship coal; and it must be gratifying to the directors and the share-

Mr. Walker. holders that the engineers had so contrived and manipulated their various movements that they were able to ship a larger quantity of coal per tip than had hitherto been shipped by any other dock or company in the world. It was modestly put by the Author, who stated that when the dock was in ordinary working trim 400 tons of coal per hour could be shipped, and if the coal could be got to the dock a great deal more—no doubt, 500 tons per hour; and with the movable tips properly brought into play, so that two or three streams of coal could be run into a ship at the same time, the power of shipping coal would be unsurpassed. It was well known that Sir William Lewis at the Bute Dock had done a great deal in that direction. The Author did not show any disposition to enter upon the question of the best way of shipping coal, especially Welsh coal, which was a very tender article. How to put it in the hold of a ship with the least amount of breakage and the least labour was a knotty and difficult question. Another source of satisfaction to the directors was that the engineers had not endeavoured to put in as much machinery as possible, but the least amount compatible with the largest amount of work, and he could state positively that there was no dock in Great Britain where the same quantity of coals could be shipped with the same expenditure of coal for driving the machinery as in the Barry Dock. Instead of taking the old-fashioned engine, cutting off steam at three-quarters of the stroke, they had adopted the best class of compound-engine surface condensers. At first sight the expenditure might appear large; but even though the first cost was greater, it was better to have compound, highly-expansive engines than to have the old-fashioned ones. It would repay any engineer or any lover of mechanical contrivances to go to the Barry Dock and see the various arrangements worked out under the direction of the men who had promoted the undertaking.

Mr. Kinipple. Mr. W. R. KINIPPLE said there was very little to criticise in connection with these works. Mr. Walker had stated that each of the high-level tips could do as much as 400 tons per hour in ordinary working. He did not wish to say that the statement was incorrect, but it certainly appeared very wonderful that with two tips going at the same time, and delivering 800 tons an hour into the same ship, that a ship of 4,000 tons could be loaded in five hours. He imagined that in ordinary working, even with the very best of loading appliances, it would be necessary to deduct a large portion of the quantity capable of being loaded per hour to allow time for trimming, probably something like one-half. Perhaps the Author was in a position to say whether a vessel

Mr. Kinippl

of 4,000 tons could be loaded in ordinary working in the time Mr. Walker had mentioned. He thought that he had done a great deal at Greenock when he designed some travelling cranes having their platforms at 22 feet above the level of the quay, carried by framing of sufficient span to cover two lines of rails. The jib head of the cranes was 76 feet above the quay-level, and had a radius of 50 feet. They had loaded twenty-six trucks in an hour, of 10 tons each, or 260 tons. More, perhaps, might have been loaded but for the fact that seldom, if ever, could an hour's work be got at any time without having to wait for the trimmers. Very often from various causes not more than 100 tons could be loaded in an hour, and then perhaps the men had to wait another hour while the trimmers did their work. The maximum ordinary working of these cranes, including stoppages from trimming, had been 1,250 tons in ten hours; or had the two cranes been loading the same ship of, say, 2,500 tons, it would have been loaded in one ordinary day; but this was a maximum, and if compared with two tips loading 8,000 tons in the same time at Barry, it was certainly more than he had ever known, and he probably could speak with some authority, as he had visited and noted the results of the working of all the best coal-tipping arrangements in the kingdom. The 20-ton Greenock cranes were built by Messrs. Appleby Bros., and are the first of a special design; others had since been built and much improved in design, especially by the addition of self-propelling gear, hauling capstans, grooved barrels for wire rope, &c. With regard to movable tips, some engineers were much in favour of them, while others considered them useless. At Greenock expensive double rails had been laid down on a jetty and two 20-ton travelling cranes provided; but after he had ceased to have the actual charge of the works at Greenock, the authorities practically fixed the travelling cranes by cutting the rails, and making a berth in the track for the wagon cradle. Who was right and who was wrong? At the Barry Docks both fixed and travelling cranes were provided, and he thought the engineers had done right in conceding to the notions of ship-owners and others, who frequently preferred one to the other. He thought on the whole that fixed lifts were the best, that was, if two could be so worked as to fit the two hatchways of one ship, and the men could do their work, for everything was handy and the moorings required no attention. The question was whether the tips were economical as regarded first cost. It appeared to him that a single tip with a tower, an incline in, and an incline out, a viaduct, and all the rails necessary to work such a tip, the cost

Mr. Kinipple. must be something like £5,000 or more. The 20-ton travelling cranes at Greenock, exclusive of the rails and jetty, cost £1,900 each; at all events they were much cheaper, and he thought less costly to work, than tips. He could not understand why tips could put so much more coal on board than cranes, when they were very similar as regarded time in their working; and besides, there was much in favour of cranes, inasmuch as the wagons could be lowered down to the very hatchways, and the coal gently emptied from the trucks by the tail tipping chains attached to the cradle, thus avoiding a great drop and the consequent breakage of coal which was so serious with many high-level tips, even although the coal was checked in its descent by stoppers. The statement by Mr. Walker that the dock gates could be held in any position at any time of tide, he thought was not quite correct. He should like to ask whether the machinery was strong enough to hold the gates in position, when there was a full head (30 feet) of water, and the gates not quite closed at the meeting posts. He thought not; at all events he should not like to attempt it. With regard to the graving-dock, Mr. Brunel had stated that he had placed safety-valves in the walls, and used very thin walls. Mr. Kinipple was not certain that those drains always kept open; he knew that they did not keep so in dock bottoms, especially in localities where the waters were muddy, for somehow or other, even although gun-metal valves having ground and watertight faces had been used, the muddy water found its way into the drains and choked them in course of time, thus rendering the walls dangerous. In most of the graving-docks that he had constructed on the Thames, such as the Regent's Dock, the King and Queen's Dock, the Prince's Dock, the Limekiln Dock and others, nearly thirty years ago, he had inserted timber drains. He called them arterial drains; they were boxes of elm with ¾-inch centrebit holes; laid longitudinally immediately under the groundways, transversely across the dock, and up the backs of the walls or behind the dock side pilings. He found that by pumping he had actually drained the neighbourhood in the vicinity of the Limekiln Dock, by lowering the permanent spring level at least 5 feet; so that after pumping a long time he had practically dried the bottom of the dock; and had destroyed most of the wells in the neighbourhood. It was a misfortune for the well-owners, but a good thing for the dock in the end. The Garvel Graving-Dock at Greenock, which he designed and carried out in 1869–73, had a gridiron bottom of timber much like crib-work in Canada, where baulks were laid transversely and longitudinally, and again transversely, and then planked. Beneath the gridiron there were five lines of longitudinal arterial drains; also drains carried up behind the walls, which were connected diagonally with the centre drains under the bottom. The main or middle drain discharged through a gun-metal valve into a sump, and as the water in the sump was kept down by pumping, so the entire dock was drained dry. Frequently during the summer months the dock at times was so dry that the people complained of the dust, which had to be kept watered to keep it down. Mr. Kinippl

Mr. A. Giles, M.P., Vice President, agreed with Mr. Kinipple that, while it was possible to load from one tip 400 or 500 tons of coals in an hour, it was not possible to tip 4,000 tons in ten hours. The great difficulty was not so much with the tips, but with the trimming of the coal in the hold of the vessel. With regard to the comparative merits of high-level and low-level tips, he had seen at the Bute Docks a movable coal-tipping machine, and it certainly was a most admirable apparatus for loading coal. It lifted the coal-truck off the rail on the level of the quay into the hold of the vessel, dropping the coal practically without breakage. It was much easier to move a machine of that sort along the quay of a dock, for tipping coal, than it was to move the ship to the machine. The first point that had struck him with reference to the execution of the work, was the closing of the bank in trying to shut out the tide from the dock. It was stated in the Paper that on the western side of the dock the bottom was hard for a certain distance from the side, and then came mud; and that the mud washed away as the tips from each side approached each other; but when it was thought the bank was safe, the tide broke through and washed a gap 80 feet wide. In attempting to close an area at the mouth of the Elbe, not so large as the one at the Barry Docks but quite big enough to cause him a great deal of trouble, when he thought he had conquered the tide it rose 6 inches higher than usual, and everything was swept away in two minutes, leaving a gap of nearly 100 feet wide. Lately, at Southampton, he had to deal with 20 feet depth of mud, and to make an inclosing-bank. He began by dredging a trench in the mud not quite, but nearly, to the bottom, and then tipping chalk into it as soon as the channel was dredged out. By that means he got the chalk to the bottom of the trench almost to the bottom of the mud, and he had very little slipping at the sides. As to the closing of a gap, where there was bad soil and a rise of tide of 36 feet to contend with, he maintained that it was bad policy to attempt to close a bank by driving in masses of soil from each end. After the painful experiences he had mentioned, his Mr. Giles, M

Mr. Giles, M.P. system was first to lay a good culvert on the bottom, even on the mud. The culvert he alluded to was not a cast-iron pipe nor a brick culvert, but a barrel of timber 3 or 4 feet in diameter made of 4½-inch plank, and hooped with iron. He had made such culverts 70 or 80 feet long, and they might be made of any length. They would stand any pressure of the bank, and when they settled in the soft mud they did not break and give. Having got in the culvert, instead of attempting to force the two banks together, or by the use of piles and shutters to close the gap, with the tide running through at the rate of 5 miles an hour, his plan would be, and he believed it was the simplest and cheapest, to leave the opening wide enough, so that the tidal scour coming through the opening should not be sufficient to scour away the soil; then gradually to fill it up 6 inches or a foot at a time, letting the tide over-ride it until the bank was high enough to be closed in one tide. A certain amount of water was shut in, but the culvert would let the water out when ready. In that way the dam could be got across without trouble or without slipping. He had done it in two or three instances. He recommended engineers, who had experienced the difficulties referred to, to adopt that method of closing a bank. It had never been his policy to make a graving-dock longer than was enough for one ship. A longer dock might be necessary in some cases; but generally it was better to have a large dock for large ships and a small dock for small ships. What had struck him in the Barry Dock was that the sill was only 24 feet below high-water of spring-tides. High-water neaps were 8 feet 4 inches below high-water springs, so that at neap tides there would only be 16 feet depth of water on the sill, which was not enough for the class of vessels at present built. He thought the sill ought to be at least 4 or 5 feet lower. With reference to the pivot of the gate, the Author had stated that, in opening, the gate relieved itself from the hollow quoin. He supposed that could only be done by an eccentric. The cup or shoe of the gate must be worked on an eccentric pivot. As the gate opened the eccentric would work it clear of the hollow quoin, and when it was shut it would take its proper bearing against the hollow quoin. He should be glad to know if that was the case.

Mr. Vernon-Harcourt. Mr. L. F. VERNON-HARCOURT noticed that the wall of the Barry Docks had the unusual feature of being without any batter, except near the base, which was certainly a great advantage in enabling ships to get alongside; and there was less overhang required in the tipping arrangements. Probably one of the reasons why batter had been generally retained was the greater stability that it

Mr. Vernon-Harcourt.

afforded to the wall, which, in the case of the Barry Docks, had been to a certain extent secured by a sharply curved batter at the bottom, by making the toe very strong (an important matter in all dock walls), and by using the best filling at the back which could be obtained. Engineers were often under the necessity of using filling which was not satisfactory; but at the Barry Docks the material was very good, and the mud of which some of the surface layers were composed had been removed from the back of the walls, so that they were not exposed to the pressure to which they would otherwise be subject. Moreover, the walls rested on solid foundations. The Author had mentioned two things which had been omitted. One was the puddling at the bottom of the dock. Judging from the strata upon which the dock was constructed, puddling at the bottom would appear to be unnecessary because even if some of the rocky strata encountered at the bottom might have been fissured, the soft mud composing the upper layers would have been sufficient to close all the apertures. Puddling was really an expensive operation, because it was necessary to take out an extra depth from the dock bottom, and then to refill with special material; and he did not think it should be resorted to unless the bottom was composed of porous material, and unless there was the risk of the water getting through and doing injury to the adjoining lands. That was the case when the Millwall Dock was first opened. The bottom of the dock was in ordinary Thames gravel, and the water percolated through into the cellars of the adjoining houses, situated on the low-lying land of the Isle of Dogs, and also into the works of the South Dock of the West India Docks, with which he was connected.[1] The other thing omitted was the supply of fresh water. All who knew that part of the coast were aware how very silty the water was that came up with the flood-tide; and in most of the docks there, and in many other places, arrangements had been made for supplying fresh water, and not allowing the muddy tidal water to enter. Probably in the case of the Barry Docks there was no fresh water available; but in other docks along the coast, as at Cardiff, Swansea and Newport, as well as at Barrow, Grimsby, and the Alexandra Dock at Hull, the expedient of supplying fresh water had been resorted to, and it was certainly very desirable when feasible. He should have thought it specially desirable at the Barry Docks, because the basin was used as a large lock; and undoubtedly, if the water in the basin was let down, as stated by

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[1] Minutes of Proceedings Inst. C.E., vol. xxxiv. p. 164.

Mr. Vernon-Harcourt. Mr. Brunel, even for a single vessel, a great deal of silty water must come in with the tide in order to raise the level; and he imagined that there must be considerable silting in the basin from that cause. But perhaps there was no fresh water to be obtained; and the silt would have to be dredged out as it was deposited. Moreover, it appeared from the plans, that the construction of a lock alongside the basin was in contemplation. It was also mentioned that there were no sluices in the walls of the passage and of the entrance. Apparently the sluices in the gates had been supplemented by a culvert at the back of the walls, so that there were other means of emptying the basin than the sluices through the gates. With regard to the closing of the dam, which was necessarily a matter of difficulty, the same thing had been attempted at the dockyard extension works at Chatham. In the first instance it was not successful, for the tide coming in and out scoured a wide breach through the embankment; but eventually, owing, no doubt, to the comparatively small rise of tide in the Medway, the dam was closed by simply tipping the material rapidly into the gap as the tide rose. The plan of closing the dam eventually adopted at the Barry Docks appeared to be similar to that adopted by Sir John Hawkshaw for closing the dam to exclude the tide from the Middle Level Drain, after the bursting of the St. Germains' sluice, which shut out the sea from the Middle Level Drain.[1] With reference to the dock-gates, the rise of the gates, being one-quarter of the span, was about the ordinary rise, and the one which was found most convenient, though perhaps not the most economical with regard merely to the gates. He was surprised at the great width at the centre of the gates. The heel-post was 2 feet 6 inches in diameter, and the central width of the gates 8 feet. That was very unusual, and he should be glad to hear the reason for that great width. It was curious to notice how some engineers adopted iron and others wood for gates. At the Alexandra Docks at Hull, wooden gates were employed[2]; at the Barry Docks, iron gates; at Liverpool, wooden gates; and he had recently had the opportunity of seeing the ribs and posts of the wooden gates which were being made for the Manchester Ship-Canal locks. At Havre there were formerly wooden gates across the big entrance of the Eure Dock, of 100 feet span; but iron gates had been substituted. A very good plan had been adopted for moving the gates at the Barry Docks.

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[1] Minutes of Proceedings Inst. C.E., vol. xxii. p. 497.

[2] *Ibid.*, vol. xcii. p. 153.

He had seen a similar plan at Antwerp many years ago, with smaller gates, without hydraulic machinery. Precisely the same system, though on a smaller scale, was in operation for opening and closing the Bougival lock gates, on the Lower Seine, by hydraulic power. He was glad to find the method adopted, because, writing five or six years ago, he said, "Moving dock gates by attaching the piston of a hydraulic engine to the back of each gate would be a great improvement, both in simplicity and rapidity, on the ordinary system with chains."[1] Chains had the disadvantage of diminishing the depth of the sill, because when the gates were open the chains had to cross one another upon the sill of the entrance. The plan of movable tips seemed to be a very good one. It had been adopted elsewhere. About a month ago he had seen the movable tips at the Alexandra Dock at Newport, where also there was a system of fixed tips. It was satisfactory to find, with regard to the Barry Docks, that in spite of the large dock accommodation at Cardiff, Penarth, Newport, and Swansea, they had succeeded in attracting a large amount of traffic. Whether some of it had been taken from other docks it was difficult to say; but it was satisfactory, in the present state of depression of the London dock companies, to find that there was scope for dock extensions in other places, and apparently without material damage to other ports, since at the Alexandra Dock at Newport a new lock and a dock extension were in progress, though probably at a less rapid rate than if the Barry Docks had not been constructed.

Mr. Verno
Harcourt.

Mr. ROBERT CAPPER said it was stated in the Paper that the centre of the entrance sill being so much lower than the bottom of the basin, a channel had been made from the entrance gates to the gates into the dock a foot deeper than the ordinary bottom of the basin. If that was so, he thought it must be a source of danger with a quickly falling tide, with only one pair of gates. As to the coal-shipping appliances, it was said that 400 tons could be shipped in an hour. In the same way it was said that dredging cost 2½*d.* a ton, and the amount came up to 2*s.* 6*d.* What kind of coal was shipped, and was allowance made for detentions? If a rate of 400 tons an hour was maintained, the collieries could not supply the quantity that would be required. A great deal depended upon the trimmer. He had watched and reported upon coal-shipping appliances in various parts of the country, and believed that no greater speed than 100 tons an hour could be maintained anywhere. He had known 2,800 tons shipped in five hours by the Lewis-Hunter cranes, but of course

Mr. Cappe

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[1] "Harbours and Docks." L. F. Vernon-Harcourt, p. 451.

Mr. Capper. that could not be kept up. It had been stated that with those cranes 10 tons had been loaded every three minutes; he had spent hours in timing them at that speed.[1] Attention had been called to the sloping sides of the dock. For ten or fifteen years he had been interested in repairing works on the banks of the Thames. For a long time the authorities were puzzled at finding so many vessels coming in with their stern frames broken without apparent cause; but on inquiry it was found that this happened only in the case of vessels that had loaded in docks with sloping banks. He had himself seen a vessel sink in the Northumberland Dock on the Tyne; the vessel had been loaded under the tips, and the stern-post caught on the sloping bank, causing its loss. Those sloping sides, he thought, were a serious drawback, and their danger had not yet been realized. In answer to Mr. Vernon-Harcourt, he might say, that all the iron gates he had known sooner or later got filled with mud, tore up the roller-paths, and gave much trouble. He had yet to see the pair of iron gates that would answer the purposes for which they were put up.

Captain Moriarty. Captain H. A. MORIARTY, R.N., C.B., said that, leaving engineering details to others better qualified than himself, he would make a few general remarks from a nautical and a shareholder's point of view; in that way he regarded the Barry Docks as a most decided success. Every natural advantage had been utilized, the site being, of course, the most important. In its adaptation to the purposes for which it was designed, the work had fulfilled more than its promoters had expected or promised. The tide outside the harbour allowed the dock to be worked for a longer period than in most other places. The stream of flood was deflected by the projecting point of the island, and he could say from experience that for fully half an hour after high-water by the shore there was slack-water for vessels to go in and out, and any person in charge of a ship in an ordinary gale could take the ship in if a pilot were not at hand. He did not advocate dispensing with the pilot's services, but sometimes when he was most needed he could not be obtained. On leaving, a ship could safely discharge the pilot within five minutes after leaving the gates of the basin. The same thing could not be said of Swansea or Cardiff. Indeed there was no port along the coast at all comparable with Barry. He [illegible] place where deep water was so close to the entrance; and the high [illegible]ection to the ships, so that they could

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[1] "The Engineer," 18 Au[illegible]26, s.s. "Glencoil" loaded 10 tons every three minutes.

Very g[illegible]
No doubt

Captain Moriarty.

go in and out with facility. He had been seven years in a dockyard, and was familiar with the working of gates, but he had no conception of such an efficient mode of opening and shutting gates as that of the hydraulic ram. He had seen nothing like the rigidity with which it held the gate, and the certainty of its operation, and it should be imitated in all future gates. From a shareholder's point of view, it was only necessary to say that it commenced to earn a substantial dividend directly it was opened.

Mr. Redman.

Mr. J. Fell Redman, holding the double office of Marine Superintendent and Superintendent Engineer, was naturally interested in docks, and he thought that while the arrangements for handling ships and their cargoes seemed to be pretty well provided for, the dry-dock department was being neglected. He would urge the importance of a little more draught of water. He believed there was a depth of only 14 feet at some states of the tide, but with modern ships 19 feet 6 inches was a common dry-dock draught. He was once put to great inconvenience through a ship stopping in a dock drawing 21 feet 6 inches. There was generally a depth of 22 feet, but the tide fell short, and the ship in dock was neaped. For the first time he let the ship go to sea against his judgment. When coming up the river to dock, he had heard a click, and he wanted to examine the ship. The diver had reported "all fair," but the key was loose in one of the outside shafts, and on the return voyage at the Nore the vessel trailed the shaft. A hydraulic lift in a dry-dock would be better than any jib-crane, which was designed for a certain lift, and was apt to be abused.

Sir James Douglass.

Sir James N. Douglass said he had gone over the Barry Docks with the Engineers, and he had greatly admired the perfect manner in which they had availed themselves of the natural advantages of the site for a coal-shipping dock, and the complete and efficient manner in which the works had been carried out in all their details. Captain Moriarty had anticipated most of what he had intended to have said relative to the approaches to these docks. The chief point of difference between the Barry Docks, and all other docks in the Bristol Channel, was the direct and ready approach from sea, and its freedom from danger in all states of weather, and the proximity of deep water, thus rendering it possible, if found to be desirable in the future, to make arrangements for vessels to enter and leave the port at all states of the tide.

Sir James Douglass. 3½ miles outside the docks, and this vessel was provided with a powerful explosive signal for foggy weather; therefore any vessel having picked up this signal was enabled to make the entrance to the docks without difficulty. He knew of no other dock in this country better situated in this respect.

Mr. Stokes. Mr. WILFRID STOKES wished to say a word with regard to the coal tips. It appeared, from the remarks of some of the speakers, that the efficiency of the tips was such that more coal could absolutely be delivered into the hold of a vessel than could be accommodated by the trimmers, who had to see that the coal was properly placed. He thought most engineers would agree that the time had come when some mechanical trimmer should be resorted to. In olden days, when coal was more leisurely stowed in the ship, the trimmers had no very difficult task to perform; but now tips were used, he thought that there might be some simple mechanical contrivance in the end of the shoot which, by means of a short vertical shaft, would allow the coal to enter below the level of the hatch. Then, by a movable hinged sloping shoot, which could be made to revolve in a radial manner, a great number of cones could be deposited at the bottom of the ship, so that at least two-thirds of the bulk of the coal could be deposited without having recourse to trimmers at all. Some time ago he saw an invention of that kind by a youthful inventor, whose name he did not remember. There was a kind of pyramid, about the size of the hatch, which, upon being let down below the hatch, had hinged flaps which opened out all round, forming shoots which delivered the coal near the outer side of the vessel; and by means of an ingenious mechanism, the act of catching hold of the pyramid again, allowed the flaps to fall, and the whole contrivance could be lifted out of the hatch without any elaborate unhinging of the apparatus. He thought something of that kind ought to be had recourse to; he was surprised to find that, in a dock which was apparently laid out expressly for the purpose of dealing quickly with coal cargoes, nothing of the kind had been attempted.

getting over that difficulty, and it would be interesting to know what it was. Mr. Stokes.

Mr. Barry. Mr. J. Wolfe Barry said that there was very little to add to the Paper in the way of description. The mode of closing the dam, adopted in South Wales, was not, perhaps, applicable to other places; or, perhaps he might rather say, the mode of closing the dam in other places was scarcely applicable in the Bristol Channel. The engineers had there to contend with a rise of tide of 40 feet; at neap-tides it was about 10 feet less; but even that was a considerable rise. The question of racing the tide by depositing material which would raise the bank higher than the succeeding tide, was fully considered before the works were begun, but the quantity of water which had to flow in and out, from the large amount of foreshore to be reclaimed, was so great, that in order to keep the velocity down to a reasonable speed, the lengths of bank would have been too large to grapple with in that manner. The contractor, however, made an attempt to do something of the kind, but it was not successful, and the matter was eventually dealt with after the mode in which Sir John Hawkshaw closed the Middle Level Drain when the sluice gave way, that was to say, by a combination of pile work and timber panels, backed up with earth and stone. By that means a sufficiently high dam could be made at one operation to ensure safety against the next tide. A question had been asked about the adjustment of the gates in the hollow quoins. That adjustment was provided by having a sliding surface on the top of the pivot on which the gate turned, the amount of sliding being controlled by a bell-crank carrying weights, that could rise and fall; this arrangement permitted of a slight amount of sliding of the gate on the top of the pivot to enable the gate to go hard up against the hollow quoin in order to take a bearing vertically through the whole of its surface. A question had been raised about the rapidity with which coal could be tipped, and Mr. Capper had stated that, although 400 tons could be shipped in an hour, that was no measure of what could be done the whole year through. It was necessary, however, to have tips which would act rapidly when rapid tipping was possible, or the average would not reach that which was looked for at the end of the year. The speed attained with trucks, averaging about 9 tons each, amounted to forty-four trucks an hour, which was

Mr. Barry. was some practical method of trimming the coal without the employment of so much hand-labour. He scarcely liked to suggest that Mr. Stokes should become an inventor, because he well knew that inventors did not always realize what they desired; but if he would invent a good practical machine for trimming Welsh coal into ships, avoiding breakage and at the same time ensuring greater rapidity, he felt certain that, even allowing for inventors' disappointments, he would realize a large remuneration for himself, besides conferring great benefit on the coal trade of South Wales. Many people had tried their hands at such a machine, which Mr. Stokes seemed to think could be made so easily. He hoped that such was the case, but he was afraid it was not so easy a matter as some persons imagined. One of the peculiarities of the dock was no doubt the gate machines. He did not think that that mode of opening and closing the gates had been elsewhere applied on so large a scale or with hydraulic machinery, but it was right to say that like many other things it was not new; the idea originated, he thought, in Holland, where it was a very common practice to employ a boom attached to the top of the gates. By a couple of small capstans and an endless chain this boom was worked to and fro, and opened and closed the low Dutch gates easily. At the Barry Docks, which were somewhat exposed to the sea, it was considered desirable to hold the gates firmly in all positions and to be able to completely control them when there was any swell or when a wave washed against the gates. After considering many different ideas, all with the same view, the large gate machines described were eventually adopted, and it was satisfactory to find that not only was the device successful so far as engineering details were concerned, but that it also was extremely well liked by the practical men who worked the gates. The engineers of this dock had not to contend, like many others, with any of the prejudices often experienced in leaving the beaten track and adopting a new method. In the present instance the dockmaster, and every one connected with the docks, considered that the rams were the best method of opening and closing gates which had ever been adopted. It was obvious that the slack of a chain falling down either in front of the gates or behind them every time the gates were opened and shut dealing with the undulation of a swell. He therefore hoped it **Mr. Barry.** might with truth be said that although the engineers had perhaps gone a little out of the way in adopting a new mode, it had turned out to be successful. Another somewhat unusual thing which they had done was to avoid the side culverts through the walls of the entrance, or passage which had been so often adopted in other docks. He thought that cutting away the bottom of a wall just at the very place where extreme pressures had to be encountered was, to say the least of it, a very objectionable thing, and should be avoided unless there was any real necessity for that method of construction. Accordingly all the sluices had been put either in the gates themselves, or in a large culvert on the west side of the entrance and passage, and at a higher level than the sills; with regard to sluicing to the level of the sill and so removing any accumulation of mud which might take place on the sill and in the entrance channel leading to the dock, they relied upon the sluices in the gates, and they had accordingly provided about 200 square feet of sluice area which could be used when wanted. He was happy to say that that quantity of sluice-power had been tried with conspicuous success in sluicing out some deposits of mud which had accumulated in the entrance channel. Until lately a freshwater brook ran over the mud east of the dock and discharged into the entrance channel, and when freshets took place, or when there was any agitation with the wind on that portion of muddy foreshore, there had been a deposition of mud in the entrance channel. That mud had been controlled efficiently by the sluicing-power in the gates; but the cause of the deposition of the mud had now been removed by the diversion of the stream. Mr. Vernon-Harcourt had alluded to the walls having been made without any batter. The point of what economy would result from adopting battering walls as compared with the very great advantage for convenient working of both imports and exports of having a vertical wall had been carefully considered; and he was quite certain that, looked at from that point of view, a vertical quay wall should be adopted for a dock. A battering wall was exceedingly inconvenient in many ways; a space was left between the ship's side and the coping into which all kinds of things fell, it increased the necessary swing of the cranes, and was troublesome in the matter of gangways and

r. Barry. to which allusion had been made, was unnecessary in the dock, as the strata were impervious, or nearly so. With regard to the deposit of silt from the water of the Bristol Channel, he might remark that the quantity differed very much at Barry from what it was at Cardiff and Penarth. There was a great deal more mud in suspension at those places than at Barry. After a series of experiments and as a matter of experience they had come to the conclusion that there would be comparatively little deposit from the mud in the sea. No doubt some deposit would have to be removed, but not a large quantity like that which had to be dealt with at Cardiff. The question had been originally considered whether it was possible to bring any freshwater feeders into the dock, but the difficulties were so great that this idea had to be given up. It was a mistake to say that all the docks in the neighbourhood had freshwater feeders. The Penarth Docks, which were some of the most successful, had been worked for many years without any freshwater feeder, being merely dependent on the flux and reflux of the sea-water. The width of the gates at Barry from front to back had been adopted as a matter of strength and amount of flotation. There did not seem to be any advantage in adopting a less width. With reference to Mr. Capper's remarks as to the sloping sides of the dock, no inconvenience had resulted from them, although, of course, some ship might get damaged by carelessness in grounding upon the sloping side. Penarth dock had sloping sides all the way along one side, and although perhaps they were not quite so flat as those of the Barry Docks, yet a ship, if it were carelessly handled, could take a bearing upon them. Not only had no inconvenience resulted at Barry, but the slopes had proved of great advantage, because they lent themselves to what was called the scarfing of ships—that was, ships lying at an angle with the side of the dock, so that one could overlap another, by which means the dock side accommodated more ships than would be possible with vertical walls. Whether that was so or not, for a large amount of the quay side at Barry there was very little choice. It had been explained that a geological fault existed on the site of the dock, and any attempt to put a masonry wall along a large portion of

Mr. Barry. dock at Barry did not belong to the Barry Dock Company; it was made for an independent company who took the lease of the land, and the depth in question was adopted as a balance of advantages and disadvantages, looking on one side to the fact that 99 per cent. of all the ships that came into the docks were light, and on the other side to the fact that if a deep sill were put in, the Dry-Dock Company would have to do a large amount of extra pumping, besides having additional capital cost on which to pay interest. As a practical matter he believed it would be found that the depth adopted was convenient, and he did not think that any business had been refused in consequence of the depth, which after all was by no means inconsiderable, being 24 feet on the sill at high-water of spring-tides, and 16 feet at high-water of neap-tides; if it did happen, that a ship drawing more than 16 feet of water had to wait a tide or two, that was not a very serious matter, when it was remembered that the depth adopted of 16 feet at neap-tides, practically accommodated almost all the trade that had to be taken in hand. The dry-dock, he believed, was a great success, doing a large amount of business, and he would suggest, that if a new dry-dock were made, which the Barry Dock Company felt should be done in the general interests of those who used the dock, the sill should be put a couple of feet deeper, so as to provide a little greater depth of water. This would be wise from the general interests of the Dock Company; but as a financial undertaking, he did not know that that would be so wise as the depth which had been adopted by a commercial company. He had thought that perhaps some figures as to the cost of the dock and railway works might be of interest to the Institution, but in giving them he wished to guard himself from the supposition that they could be relied upon as absolutely accurate, because the accounts of the undertaking had not been completely adjusted, and some alteration might therefore be required. The dock works, including gates and caisson, had cost £620,000, or about £8,300 an acre, and about £136 per lineal yard of quayage. The equipment, by which he meant all the railway sidings round the dock, the coal-tipping machinery, the hydraulic machinery, and all matters required to make the

Barry. large dock undertakings would regard as a low figure. The railways of the Company cost about £760,000, or about £27,700 per mile. That was no doubt rather a high figure; but the country through which the line passed was by no means easy, and involved some long tunnels and viaducts. The whole railway had been finished in a first-class style, and was in every way fitted for the traffic which he supposed was or would be as heavy as that of any railway in the kingdom. The breakwaters cost about £60,000, or £60 per lineal yard; the timber gantry £24 per lineal yard; the dock walls from £48 to £52 per lineal yard. He merely gave the above figures for what they were worth, not professing to say that they were absolutely correct, but they were a pretty near approximation to the truth. During the first six months of the opening of the dock, a trade had developed which had enabled the Company to pay 5½ per cent. upon the ordinary capital; and the trade had been growing so largely, that last week the exports amounted to 75,000 tons, or approaching the rate of 4,000,000 tons per annum.

binson. Mr. J. Robinson, in reply, said that the line of the principal geological fault referred to by Mr. Brunel was shown on the plan accompanying the Paper. On the southern side of the fault was red marl, and on the northern lias, which had been thrown down 70 feet or more. He was of Mr. Walker's opinion, that the system of opening and closing dock-gates, by direct-acting hydraulic cylinders and rams attached to the gates, would be adopted in future by dock engineers. The gate machines, referred to by Mr. Kinipple, were not designed to open the gates with 30 feet head of water; they were not unfrequently opened, however, with 2 feet head, at the reduced pressure of 250 lbs. per square inch, and held the gates firmly while they were being opened and closed; they could be stopped at any point in the stroke of the ram, and their movement continued or reversed at the will of the men who manipulated the valves. There were no chains in connection with the gate machines. The compound condensing pumping-engines, commended by Mr. Walker, had been tested, and performed what they were guaranteed to do, in respect to the consumption of steam and Welsh coal per indicated HP. It was difficult to say which was the best method of shipping Welsh coal, many changes had taken place, and perhaps it was not yet known.

of the coal was tipped between that level and 10 feet below it, so that the tipping table had not to be lowered and lifted to the extent that would be required in any other position. The wagons ran straight on and off the tipping-table without requiring to be turned each time, and the shunting was but little. There were four rams under the table, so that any power, or combination of powers, could be used. The butt-end of the shoot was 11 feet wide, with high sides, and received the whole of the contents of a 10-ton wagon at once. The arrangements were thus economical with regard to hydraulic power and manual labour, and such as to ensure rapidity of motion. Since the Paper was written, trains of coal, consisting of one hundred wagons, had been brought down to the dock with two engines. In reply to Mr. Giles, Mr. Kinipple and Mr. Capper, although 440 tons had been tipped into a vessel from a single high-level fixed tip in an hour, that rate could not be maintained throughout the loading of the vessel. As an instance of what was actually done, he might take the case of the s.s. "Beethoven," which had four hatchways for the cargo, and two for bunker coal. The vessel was placed under a tip at 5 P.M. one day, and received a full cargo at 6.30 P.M. the following day; during that time 3,088 tons of cargo coal, and 338 tons of bunker coal, making a total of 3,426 tons, were loaded, being at the rate of 135 tons per hour, including all stoppages for shifting the vessel to the different hatchways, and the men's meal-times. When the movable tips were completed and in working order, and the coal was tipped in from a fixed and a movable tip at the same time, the rate of speed would be increased, but at present he was unable to say what the rate would be. For trimming coal, boards 2 feet 4 inches wide, covered with galvanized iron, and 12 feet and 6 feet long, were laid so as to form inclined planes, along which the trimmers shot the coal. When tipping bunker coal, a temporary saddle of planks was made on the raised portion of the deck, between the small side-bunker hatchways, to enable the trimmers to shovel the coal into both hatchways at the same time. Some of the steam-ships frequenting the Barry dock were called self-trimmers. They had two large hatchways with feathered wings, and there was not much more to do in the way of trimming coal than levelling down. The carrying capacity of those vessels ranged from 1,200 to 1,800 tons. There was no difficulty in moving the cranes or the ships when there were

Mr. Robinson.

obinson. rate of flow through various-sized openings; but the plan was not adopted, and the scheme carried out was for excluding the tidal water as much as possible. The cast-iron pipe culvert through the western dam was laid on a cradle of timber, and the tide, at ordinary springs, rose 28 feet above it, so that it was something like the culvert referred to by Mr. Giles. With regard to the long graving-dock being divided into two portions, ships entering for repairs occupying the whole of one division and part of another were rather exceptional. At high-water neap-tides there was a depth of 16 feet on the sill; and as the basin was used as a lock, the water in the large dock was not allowed to fall lower than would give a depth of about 15 feet of water on the sill of the graving-dock; but, should the water in the dock be allowed to fall to the level of the lowest neap-tides of the year, there would still be a depth of 11 feet 6 inches of water on the sill. That would not be sufficient for a large steamer, nor for a sailing-vessel with ballast. To have made the sills of the graving-dock 4 or 5 feet lower, as suggested by Mr. Giles, would have increased the working expenses of pumping more than was desirable, when economy of working was considered. Most of the ships entering the graving-dock were light, and drew about 11 feet of water, and those drawing 12 or 13 feet were not numerous. Should vessels of a greater draught be more used in the coal trade, it would be desirable to make a new graving-dock 2 or 3 feet lower. It had been stated that the bottom of the greater part of the dock was like puddle. At the east end there was stone and rocky marl, but the tendency was for the puddle to work its way into the crevices. With reference to Mr. Capper's question, as to whether the dock had robbed other docks in South Wales, he might mention that, since its construction had commenced, the increase in the shipment of coal from Cardiff had been 3,000,000 tons. The channel 1 foot deeper across the basin from the entrance was to allow vessels to enter so that they would not stick on the sills. There was no danger, because the moment they entered, when they came in late, the entrance gates were closed and the passage gates opened, so that the ships proceeded at once into the dock. In answer to Mr. Stokes' enquiry, it was proposed to make good the gaps in the rails of the movable approaches to the movable tips by loose sections of rails.

Correspondence.

Mr. A. C. Andros was gratified to notice that in the gate-hauling machinery, a design had been adopted which, he believed, he had been the first to suggest eleven years ago.[1] At that time he strongly condemned the appliances usually employed in working dock gates by hydraulic machinery, which were, in his opinion, antiquated, costly, and unmechanical. He was particularly impressed with this in superintending the construction of the Victoria Dock Extension for Sir (then Mr.) Alexander Rendel. There were four pairs of gates, of 80 feet span, requiring to open and shut them no less than an aggregate length of 3,680 feet, or 23 tons, of 1¼ inch chain, sixteen hydraulic cylinders and sixteen roller-boxes, and four score pulleys and sheaves besides. By adopting his plan of a direct-acting ram, all this chain would have been dispensed with, only eight hydraulic rams would have been required and no roller-boxes, pulleys or sheaves would have been necessary, thus achieving a very large saving in original cost, maintenance, labour in working and expenditure of hydraulic power, besides a much better method of working. He perceived that in Barry Dock entrance the design he had illustrated, in the Minutes of Proceedings had been pretty closely followed, the only deviation being that the rams had been placed in the centre of the gate recess laying hold of the gate in the centre. This was no doubt a more effective point of attachment than nearer the heel-post as shown in his plan; but he would prefer tucking the ram as close as possible into the corner between the gate and the recess, first because it reduced the risk of any barge or other craft drifting behind the gate and fouling the ram, and secondly, because it reduced the length of stroke of the ram itself, both objects of considerable importance. He felt satisfied now that this system had been adopted by the engineers of the Barry Dock, and pronounced "very successful," their example would be followed by other engineers, and that the old-fashioned hauling gear would be relegated to the limbo of the past. Mr. Andros.

Mr. Killingworth Hedges remarked that the system of illumination which has been adopted, namely, of fixing numerous arc lights on poles 40 feet above the quay level, had certain disadvantages over what might be termed the "beacon" plan; this consisted of a few powerful arc lights placed on towers not less Mr. Hedges.

ledges. than 75 feet in height, the towers being spaced at such a distance apart as would cause an equable distribution of light over the entire area. He had introduced this system for the illumination of the Langton and Alexandra Docks at Liverpool in 1881, and the same method had, after careful investigation, been adopted for other portions of the docks. The towers were made of light latticework 75 feet high, 2 feet 6 inches square at the base, and 10 inches square at the top; they were firmly fastened to a massive cast-iron base, so that no guy lines were necessary. The lantern was hoisted by means of the electric cable which was run over an insulated drum; electrical connection was effected at each end by copper brushes. The Serrin regulator was still used, and although this was one of the earliest arc lamps, it could not be excelled for rough work. Each "beacon" was connected to a Gramme dynamo of the old A type, giving a light of 6,000 candles nominal and an illumination sufficient for all dock purposes for a radius exceeding 100 yards. Some of the dynamos used on this installation were probably the oldest in the country, having been worked almost nightly since 1877, when electric light was first used at the Liverpool Docks for the construction of the Langton Dock. The "beacon" system was cheaper than that described by the Author; there were fewer lamps to keep in order, therefore the attendance for re-carboning and cleaning was minimized; as a rule the length of cable required was less, and it could therefore be economically laid underground. The lamps were much simpler than those placed in series, and what was a still greater advantage, the working pressure was low; therefore no serious danger would ensue from the fall of an overhead wire.

lurtzig. Mr. A. C. Hurtzig observed that the enormous quantities of coal being shipped in South Wales, something like 1,000,000 tons per month at Cardiff alone, not only required that the coaling-appliances at a new dock should be of the most approved kind, but also that every arrangement should facilitate the manipulation of ships and tend to save time. A basin, 7 acres in extent, which was the only lock, was in his opinion a serious defect, and the sooner the "proposed lock" was constructed the better he thought it would be for the proprietors and for the reputation of Barry Dock. The majority of steamers taking coal cargoes would have draughts under 22 feet, and if a steamer through no fault of its own had been delayed in taking cargo, and required to leave the dock two

Mr. Hurtzig.

for demurrage? If there were a moderate-sized lock the operation could be conducted without difficulty, but, with a 7-acre dock, the operation would take nearly an hour, and the steamer would leave the harbour at about half-tide, when the conditions would be most unfavourable for navigation. At the same time, an unnecessary amount of water would be run out of the dock, aggravating the disadvantage of the shallow depth upon the graving-dock sill. If there were no necessity for steamers to approach or leave at such times of the tide, where was the value of so deep a sill as 38 feet to the lock? Steamers might, truly, pass out in fleets through the basin-lock, but they could be despatched one or two at a time through a small lock with less loss of water and quite as expeditiously. The coaling-appliances at Barry were very good, but it was a question whether this system of coaling was the best that could have been adopted in laying out a new coal port. The performance of shipping 400 tons per hour "in ordinary working" (p. 87) was, no doubt, an extraordinary one, judging from the capabilities of similar appliances at other ports. In 1887, as Engineer of the Hull and Barnsley Railway and Dock Company, he had to visit a number of coal ports with the special object of examining the different systems of coaling, so that the directors might have before them the latest and most approved methods previous to deciding upon an extension of their own coaling arrangements at Hull. The ports visited were the Newcastle-on-Tyne, Hartlepool, and Grimsby, on the east coast, Swansea, Newport, and Cardiff, in South Wales, and Liverpool. The result of the investigation was that the directors decided that the system already introduced at Hull by Mr. Abernethy, Past-President, namely, the Newport system, was that which would give the best results both as to economy and to expedition under the ordinary every-day conditions of working. At Barry the systems were similar to that at Newport, but the modifications in his opinion were not improvements. The Newport method and that at Hull was briefly this:—The collecting sidings for full wagons were on a raised embankment, which might be 5 to 8 feet high, according to their distance from the quay side. From these sidings there was an incline of 1 in 100 down to the turn-table on the quay-level. On this incline were three roads, so that a large number of full wagons might be collected at each tip. These three roads converged to one road on the turn-table, which was only a few feet distant from the table of the coal-hoist; the wagons were let down one by one on to the turn-table without engine power, and thence were hauled by a hydraulic capstan on to the hoist. The wagon was

Mr. Hurtzig. next lifted to the required height for tipping into the hatch of the vessel, and when empty was brought back to a level about 14 feet above the quay. It was then pushed off the hoist on to a viaduct, which was at a gradient of 1 in 75, and which allowed the wagon to run down, by its own gravity, on to the siding from which the train of empties was taken away. The hoist in the meantime had been lowered to quay level, and had taken another full wagon up for tipping. In this system no one operation interfered with another, and there was less liability to interruption and delay than in any other process with which he was acquainted. It was evident, other things being equal, that the system of low-level tips at Barry could not compete with the Newport system for expedition. The high-level tips were, however, superior to the low-level ones; but the gradient for full wagons, 1 in 233, was too flat to allow the wagons to be moved easily without power, and there was a length of road adjacent to the tip, common to both full and empty wagons, which was likely to cause interruption and delay. With regard to the 400 tons an hour, stated to have been shipped at Barry, he had seen at Hull Alexandra Dock, in ordinary working, a wagon of 10 tons brought on to the hoist, tipped, pushed on to the empty road, and a second full wagon brought ready for tipping in a little over a minute and a half. This was equivalent to about 400 tons an hour; but he had never known of a ship loaded at Hull at so much as half that rate. The quickest loading he had seen was that done at the Tyne Dock of the North Eastern Railway Company. Here there were no hoists, the wagons were brought on to the tipping-staiths at a level higher than was required for any vessel. The wagons were all hopper-bottom wagons, and occasionally a vessel could take coals at two hatches from two spouts at the same time. The following were recorded rates of shipment: s.s. "Beckton," on the 11th of September, 1880, loaded with 1,868 tons of coal in four hours and a half, two spouts working most of the time; s.s. "Great Yarmouth," on the 5th of May, 1881, 1,066 tons in two hours and a quarter, working two spouts; s.s. "Spero," on the 23rd of March, 1882, 1,044 tons in two hours and ten minutes, working two spouts. In each of these cases the time given included the time occupied in bunkering coals for the steamers' own use. At Cardiff, tipping by the balance-tips occupied three minutes and a half per wagon, the maximum work having been seventeen wagons per hour. The greatest quantity of coal shipped in one year at one of these tips had been 450,000 tons, equal to about 9,000 tons per week. At the low-level tips at

coals could be shipped at the rate of 300 tons per hour;

Mr. Hurtzig but the average speed was 80 tons per hour. At the Roath Basin he had seen the operation completed in a minute and a half per wagon, but the average rate did not exceed sixteen wagons per hour. At Newport he had been informed, in 1887, that 2,400,000 tons of coal had been shipped at eight hoists in twelve months, being about 6,000 tons per hoist per week over the whole year. These hoists were capable, for short periods, of shipping at the rate of 300 tons per hour. It was thus evident that at the principal coaling ports the speeds at which cargoes were loaded was governed by other conditions than the perfection of the coaling machine, such as rate of trimming the coal in the ship's hold, and the facilities for bringing down full wagons and for removing empties. At the Tyne these facilities were very complete, yet while it was possible to ship down one spout at the rate of 500 tons per hour, it was considered very good work indeed if a cargo could be shipped at this rate with two spouts in operation. It would be the same at Barry, the 400 tons per hour would be no criterion by which to form a judgment of the average capabilities of the coaling arrangements, and as the method of bringing full wagons down and taking empties away might possibly allow of one of these operations interfering with the other, he anticipated that the average performance would be inferior to that at Cardiff or Newport.

Mr. Kidd. Mr. W. Kidd remarked that in connection with the passing of the Bill for the construction of the Barry Dock, he was, in 1884, instructed to make a series of observations and tests of the speed of shipping coal at a number of the leading Scotch coal-shipping ports. It had occurred to him that the information thus obtained might be of some value in considering the coal-shipping appliances with which the Barry Dock had been equipped, as described by the Author, and which he took it were illustrative of the best modern practice in this kind of machinery. He had accordingly abstracted the results, and desired to submit the following Table to the Institution. The appliances at all the ports included in the Table were upon the low-level system, and, with the exception of Glasgow, the docks were of such a size as to accommodate only vessels of moderate tonnage. The results were obtained in the ordinary course of working, no special preparations having been made with a view to speed tests, and they might be taken as fairly representing the ordinary work of the machinery. It, however, happened that in no case was there any stoppage for trimming, as in some cases the vessels were "self-trimmers," and in the others the "cone" below the hatchway was being formed when the speeds were observed. Table No. I showed that Armstrong's hoists at

Mr. Kidd. Grangemouth Dock, Example No. 5, gave the highest results as to speed, while Armstrong's cranes at the Queen's Dock, Glasgow, Example No. 1, were the slowest, the former shipping at the rate of 406 tons per hour, and the latter at the rate of 144 tons per hour, or little more than one-third. Thus were speed of shipment the only consideration, cranes would not be able to hold a place. In all except No. 1, the appliances were of the same type, namely: hydraulic hoists, but while at Grangemouth the rate of shipment was 406 tons per hour, the next highest by hoists, Burntisland Dock, Example No. 2, was only 264 tons, and the lowest, Burntisland Dock, Example No. 4, only 160 tons per hour. While an examination of the Table showed that there were considerable differences in the speeds of the mechanism (Table III), heights of lift, &c., it would be found that the great saving of time in the "intervals" between the wagons at Grangemouth was the principal reason of the very high speed obtained there, as was clearly shown in Table II. The saving of time was effected chiefly by the high-level line for taking off the light wagons, and partly by the self-acting gradients for bringing on and taking off the wagons. This arrangement, which had been adopted in none of the other cases, enabled a loaded wagon to be brought forward, turned, and placed in position close up to the hoist-framing with the capstan-rope attached, all ready for placing on the empty cradle as soon as it reached the quay level, while the previous wagon was being raised and emptied. The higher speed in the return strokes in the Grangemouth hoists, another considerable source of saving of time, was obtained by sending the return water to waste into the dock instead of to the engine-house tank, which was a source of considerable delay in Nos. 4 and 6. The self-acting gradients also effected a saving; after the loaded trains had been shunted into the standage lines by a locomotive, nothing further was required from locomotive, horse, or capstan, except to pull the loaded wagon off the turn-table on to the cradle, a distance only a little more than its own length. They were thus economical, and they proved exceedingly handy here as at all other places where he had seen them employed, especially in the North of England high-level system. Perhaps in no district were steamers loaded more rapidly than in the north-east of England, under the system of high-level spouts. Those at Whitehill Point on the River Tyne afforded a good example of this system; they were designed for the largest class of steamers. Each staith there was provided with three spouts so as to load at three hatchways of a vessel simultaneously, and they were capable of loading at the rate of from 800 to 1,000 tons per hour. The spouts were 45 feet apart from centre to centre; but each spout could be swung to the extent of about 15 feet on either side of its centre to suit the hatchways of different vessels. The rail-level at the spouts was 36 feet above high-water of ordinary spring-tides, or 51 feet above low-water of ordinary spring-tides, and seven spouts were provided to any one of which the movable shoot was attached, according to the level of the tide and the height of the ship's deck, which latter might vary between about 15 feet and 35 feet below the rail-level. The wagons carried only 4 tons each and opened at the bottom; they were brought to within 540 yards by locomotives, and were thereafter worked by self-acting gradients. He had alluded to this system principally to show that while steamers were very rapidly loaded by it, this rapidity was chiefly due to the simultaneous loading at more than one hatchway, which method was universal in that district, whereas at the Scotch ports only one hatchway was in use at one time. Were three hoists like those at Grangemouth Dock applied simultaneously to the loading of a steamer, the speed would exceed 1,200 tons per hour on the actual results obtained there, and were wagons of 10-ton loads employed in place of wagons of 7 tons capacity, this would be further increased. But, similarly, by enlarging the very small capacity of the wagons of the Northumberland district, the speed could be largely increased, so that he thought the spout system with hopper wagons would be likely to maintain its supremacy in the matter of rapid shipment. Of course, all the statements as to the speed of shipment depended upon there being no stoppages for trimming, which, as a matter of fact, frequently took place at all the ports named, and chiefly so with the smaller and older types of vessels. He thought, however, that shipbuilders were now able to turn out well-deck steamers for the coal trade that were practically "self-trimmers." Besides attaining a high speed of shipment, however, it was of great importance that there should be as little breakage of the coal as possible. Of the examples in the Table, the crane system at Glasgow, Example No. 1, gave the best results as regarded minimum breakage; and seeing that it enabled the wagon to be placed in the hatchway before breaking bulk, it surpassed all other systems of shipment. In all the other cases mentioned in the Table, the coal first fell from the wagon into the shoot, a drop of from 3 to 4 feet, then slid down the shoot about 25 feet, and finally fell into the hatchway. By the Tyne spout system the coal first fell some 3 to 5 feet into the spout, slid down the inclined spout and shoot, a distance varying from 35 to 75 feet, and thence it fell into the hatchway. While none of these systems enabled the fall of the coal from the hatchway into the bottom of

Mr. Kidd.

Mr. Kidd.

TABLE I. LOW-LEVEL SYSTEM OF SHIPPING COAL.—

Example.	No. 1. Queen's Dock, Glasgow.	No. 2. Burntisland Dock. No. 2 hoist, first test.
Kind of machine	20-ton hydraulic crane, having small oscillating rams geared to cupped sheave, with cradle for lifting and slewing wagon over hatchway.	15-ton hydraulic coal-hoist, with direct lifting and tipping rams under cradle. Empties coal into a movable shoot.
Makers	Sir W. G. Armstrong and Co.	Sir W. G. Armstrong and Co.
Railway sidings	All level; wagons worked by capstans.	All level; wagons worked by horses.
Speed table (each operation)—	Mins. Secs.	Mins. Secs.
Lifting wagon	0 41·0	0 13·25
Slewing „	0 27·0	..
Tipping „	0 25·4	0 14·5
Slewing back and depositing wagon	0 35·3	..
Lowering wagon[3]	..	0 19·75
Removing light wagon, and turning and placing loaded wagon on cradle	1 18·0	1 2·5
Total time per wagon	3 26·7	1 50
Actual number of wagons shipped per hour, including stoppages	18	33
Load per wagon	Tons. 8	Tons. 8
Actual shipment per hour	144	264
Level of coaming of ship's hatchway	5 feet above quay.	Quay level.
Height of lift of cradle	About 8 feet.	7 feet.
Men employed at each machine	5 men, 1 capstan.	6 men, 2 horses.

Notes to Table I.

In none of the above tests was there any waiting on account of trimming.

All the wagons were end-tippers, wagons open at one end only.

A test of No. 3 hoist, Burntisland Dock, with height of lift of 13 feet, gave same result as Example No. 3, under less favourable conditions.

Examples Nos. 2, 3 and 6 loading spar-decked steamers; all others well-decked steamers.

In No. 4 the arrangement of rails more favourable than in Nos. 2 and 3 at same dock.

In Nos. 1 and 6 most of wagons had to be turned through a semicircle.

In Nos. 2, 3 and 4 all wagons had to be turned through a quadrant.

In No. 5, all wagons had to be partly turned.

Mr. Kidd.

RATES of SHIPMENT with DIFFERENT MACHINES.

No. 3. Burntisland Dock. No. 2 hoist, second test.	No. 4. Burntisland Dock. No. 4 hoist.	No. 5. Grangemouth Dock.	No. 6. Bo'ness Dock.[1]
Same as No. 2	15-ton hydraulic coal-hoist, with two overhead lifting rams attached to cradle by wire ropes, and tipping-ram under cradle. Empties coal same as No. 2.	15-ton hydraulic coal-hoist, with direct lifting and tipping-rams under cradle. Empties coal same as No. 2. 27 feet lift.	Same as No. 4.
Same as No. 2	Brown Bros. and Co.	Sir W. G. Armstrong and Co.	Brown Bros. and Co.
Same as No. 2	All level; wagons worked by capstan and horse.	Self-acting gradients; wagons worked by gravitation.[2]	Self-acting gradients; wagons worked by gravitation and capstan.
Mins. Secs.	Mins. Secs.	Mins. Secs.	Mins. Secs.
0 25·4	0 40·8	0 20·33	0 14·6
..	..	..	..
0 25·2	0 43·7	0 14·83	0 28·3
..	..	..	..
0 23·2	0 54·8	0 14·22	0 46·2
1 6·4	0 40·7	0 12·5	1 0·4
2 20·2	3 0	1 1·88	2 29·5
26	20	58	24
Tons. 8	Tons. 8	Tons. 7[4]	Tons. 8
208	160	406	192
8 feet above quay.	5½ feet above quay, and 7½ feet.	7½ feet above quay.	1½ foot above quay.
15 feet.	13 feet and 15 feet.	14½ feet,[5] and afterwards up to light line.	8½ feet.
6 men, 2 horses.	5 men, 1 boy, 1 horse. 1 capstan.	6 men, 1 capstan.	5 men, 1 capstan.

Notes to Table I.

[1] These hoists are described and illustrated in Minutes of Proceedings Inst. C.E., vol. lxx. p. 369, and Plate 15.

[2] Gradient of load lines 1 in 100; light line 1 in 80. Wagons left hoist at 15 feet above quay level.

[3] "Lowering" included back stroke of tipping-ram.

[4] Taking this at 8 tons, as in all the other examples, would give a shipment 464 tons per hour.

[5] Quay level was too low—3½ feet above high-water of ordinary spring-tides, necessitating increased height of lift.

Mr. Kidd. the hold to be avoided, which could only be provided for by using some system of anti-breakage appliance for forming the "cone," it was obvious that much breakage, previous to the coal reaching the hatchway, was avoided by the crane. With a view to retaining the great advantage of cranes in the matter of minimum breakage, he had been led to suggest a mode of increasing the speed of shipment by employing what he might term a "double-jib crane." It was a development of the design of the cranes employed at the Roath Dock, Cardiff, designed by Messrs. Sir W. G. Armstrong, Mitchell & Company, and it should be provided with the same ingenious auxiliary appliances. It should be movable, travelling on wheels, but resting on four hydraulic jacks while working. It would therefore have to be provided with two of Westmacott's coaling cradles, which enabled wagons to be lifted and deposited at any point on a line of rails. It should also have the swivel arrangement, which enabled the wagons to be turned while suspended by the crane. At the Roath Dock it was found that a wagon could be shipped in from two minutes and a half to three minutes. By the "double-jib crane" two wagons could be shipped in that time, as, while at one end the wagon was being emptied into the hatchway, at the other the light wagon could be taken off and a loaded wagon placed on the cradle. In addition to a second jib and cradle, only one second set of lifting- and tipping-cylinders would be required. The distribution of strains in the crane would be good, the difference in the counter-balance being only the weight of the coal in the wagon. Two men would be required to work the machinery. He proposed to apply two such cranes to each moderate-sized vessel, and three to each large one. Taking the wagon loads at 8 tons, and allowing the same time for each "revolution" as at the Roath Dock, two wagons would be shipped in two minutes and three-quarters, which gave 348 tons per hour for each crane. In other words, a moderate-sized vessel could be loaded at the rate of 696 tons, and a large one at the rate of 1,044 tons per hour. An important advantage, resulting from the use of movable cranes, was that each vessel required but little more than one-half the length of quay necessary with a fixed machine. In the former case it was not necessary to move the vessel, in the latter a 300-foot ship required a length of 550 feet, in order to shift ahead and astern, so as to bring the hatchways under the shoot, where only one machine was employed to each vessel, as at Glasgow. The last point he had to mention was that, at Glasgow, the cranes were frequently employed in the shipment of machinery, &c., which gave them a further advantage over other coal-shipping appliances.

Mr. Kidd.

TABLE II.—TOTAL TIME WAGON was on CRADLE COMPARED with the INTERVAL BETWEEN WAGONS.

—	No. 1.	No. 2.	No. 3.	No. 4.	No. 5.	No. 6.
	Mins. Secs.	Mins. Secs.	Mins. Secs.	Mins. Secs.	Mins. Secs.	Mins. Secs.
Average time of wagon on cradle .	2 8·7	0 47·5	1 13·7	2 19·3	0 49·38	1 29·1
Average interval .	1 18·0	1 2·5	1 6·5	0 40·7	0 12·5	1 0·4
Total time per wagon	3 26·7	1 50·0	2 20·2	3 0	1 1·88	2 29·5

TABLE III.—AVERAGE SPEED of LIFTING and LOWERING CRADLE with LOAD.

—	No. 1.	No. 2.	No. 3.	No. 4.	No. 5.	No. 6.
	Mins. Secs.	Mins. Secs.	Mins. Secs.	Mins. Secs.	Mins. Secs.	Mins. Secs.
Lifting, 1 foot in	0 5·12[1]	0 1·9	0 1·7	0 2·9	0 1·4	0 1·7
Lowering,[4] 1 foot in	Included in back slewing	0 1·52	0 1·1	0 2·74[2]	0 0·70[3]	0 3·19[2]

Notes to Table III.

[1] Low speed due to gearing between engine and chain sheave, and to multiplying sheaves.

[2] Very slow. Caused principally by tipping-rams having to force return water back to engine-house tank.

[3] Higher speed obtained by running return water to dock.

[4] A height of 6 feet is added to that of the lift for the back stroke of the tipping-rams.

Mr. Robinson.

Mr. J. ROBINSON, in reply to the correspondence, observed that the beacon plan of arc lighting advocated by Mr. Hedges, would be unsuitable for the Barry Dock Works, where the lines of railway were on different levels, and all the iron framing of the coal tips was standing up 60 feet above quay-level in various places. A concentrated form of light would be objectionable on account of the shadows, particularly in the coal tips, where lamps had to be hung inside, to overcome this difficulty. Mr. Hedges lighted an area of 100 yards radius, with a 6,000 candle-power lamp, on a 75-foot pole; but the open parts at Barry, such as the high-level sorting and marshalling sidings, were efficiently lighted by about the same number of lamps of half the nominal candle-power, and the system gave satisfaction. On the question of cost, supposing half the number of beacons were employed, and a dynamo to each, Robinson. although there would be only half the number of lamps to keep in order, there would be ten times the number of dynamos and twenty times the number of wires; and a plant divided into such small units could not be so efficient as a less sub-divided one. The arrangement of circuits at Barry did away with the danger of having two wires of widely different pressures near one another, the circuit being a complete circle round the dock. Mr. Andros's illustration of a direct-acting gate-ram represented, in outline, pretty well the design worked out at Barry. The rams at the passage differed as to position and attachment from those at the entrance. At the passage the ram was placed nearer the heel-post, not at the centre of the gate-recess as at the entrance, and the attachment was 14 feet 6 inches from the heel-post; whereas at the entrance it was 25 feet, or half way between the heel- and the mitre-posts. The stroke of the ram at the passage was 16 feet 7 inches, and at the entrance 25 feet 9 inches. Furthermore, the ram at the passage was attached to the gates near the top, whilst at the entrance it was 11 feet 3 inches below the timber decking or quay-level; the water-level in the basin being frequently lower than in the dock permitted of this being done, thus allowing the ram to be examined and repaired when out of water. When the gates were closed, wrought-iron box-booms were drawn across the passage and entrance to prevent craft or anything else from fouling the rams or injuring the gates. In reply to Mr. Hurtzig, a steamer, drawing 22 feet of water, wanting to leave the dock two hours after high-water of a spring-tide, under the circumstances mentioned, would be passed through the basin; and the operation would take, instead of an hour, between ten and fifteen minutes, from the time the ship left the tip, until it entered deep water; indeed, traffic, as a rule, was worked two hours before and two hours after high-water. Vessels drawing 21 feet had been let out two and a half hours after high-water of a neap-tide; and in exceptional cases, vessels had been let out at half-tide, or three hours before and three hours after, high-water. Notwithstanding all this traffic, the minimum depth of water was 30 feet in the dock. The sluices lowered the water in the basin at the rate of 1 foot per minute, and 1 foot in the basin was 1 inch in the dock. A lock formed part of the original plan for the Barry Dock, and the intention of constructing it had not been abandoned. With regard to the Newport system of coaling, the full wagons had all to be lifted from quay-level to the necessary height to be tipped, and the empty wagons lowered to 14 feet above quay-level to leave the hoist; whereas the reverse of this operation was carried out at

Mr. Robinson.

the high-level tips at Barry; all, or nearly all, the wagons were lowered full, and lifted when empty. With the Newport system after the empty wagon had left the hoist, it had to be lowered to quay-level to receive a full one; but at Barry the hoist was in position to receive a full wagon, and no sooner was an empty wagon pushed off than a full one could be drawn on at once. The level of the rails where the wagons were received on to the cradle was 31 feet 4 inches above high-water of ordinary spring-tides. The piece of road adjacent to the tip, common to both full and empty wagons, was 5 yards in extent, a length sufficient to afford standing-room for the wheels of a wagon between the hoist and the self-acting points, and the distance from the latter to the self-acting crossing was 12 yards. That the gradient of 1 in 233 was not too flat was shown by the fact that the hydraulic wagon capstan drew fifty-six wagons down it. From actual experience, all the arrangements at the high-level tips were found to be satisfactory; and quite recently, 490 tons of coal had been put on board the s.s. "Swainsby" from a single fixed tip in an hour, without any special preparations having been made, and as much as 300 tons had been tipped in half an hour. The Barry system of high-level tips was superior to the low-level Newport system, and the latter was superior to the Barry low-level. With regard to the best system to be adopted for laying out a new coal-port, that would depend much on the configuration of the ground surrounding the dock, as to whether a high, low, or intermediate level could be carried out. As mentioned by Mr. Kidd, the most rapid loading was perhaps done on the River Tyne, where the staiths were provided with spouts, so that vessels in some cases might be loaded through three hatchways simultaneously; but the hopper wagons in use there were so small they would not allow the large lumps of Welsh coal to drop through. Respecting the utilization of quay space, by means of a movable tip in connection with a fixed tip, it would not be necessary to shift a vessel so far ahead or astern to bring the hatchways under the shoots as with single fixed tips, and the sloping sides enabled the ships to keep the position of echelon alongside the quay.

8 April, 1890.

This being the Tuesday in Easter week, in accordance with the By-laws, there was no meeting.

SECT. II.—OTHER SELECTED PAPERS.

(*Paper No. 2381.*)

"Concrete Quarters for Native Clerks, Guards and Menial Staff on Indian Railways, &c."

By THOMAS KER, M. Inst. C.E.

THE excessive cost of housing the native staff, owing to the necessarily, and by comparison with European railways, large number of native employees required, has long occupied the serious consideration of railway administrations in India. The above difficulty, in effect, has led in many instances to the resort to temporary structures, the repairs of which cost, in a few years, more than permanent quarters could have been erected for.

The Author, having had some experience in the designing and construction of various descriptions of dwelling-houses in India, has, with the object of meeting the exigencies of the case, designed a class of quarters suitable for natives, including clerks, guards, platelayers, pointsmen, gatemen, and the lower staff generally, for which he claims the following advantages.

1. The quarters being built of concrete throughout, foundations, walls and arched roofs, can be erected almost anywhere.
2. The materials, lime, sand, broken stone, or brick or kunkur, are nearly everywhere procurable locally.
3. Very little skilled labour is necessary.
4. The cost is one-half that at present paid for the same area of stone or brick-built accommodation.

The quarters are watertight, and, if properly constructed, should never require repairs; those erected some years ago have withstood every test. The natives prefer the above as being cooler than stone-built houses.

The style of construction adopted is specially suited to the natives, who are accustomed to light work, progressing by easy stages. It also has this advantage that, with the usually scanty supervision available in India, the due inspection of each course laid is rendered possible, there is less opportunity of inferior materials being passed, and a better guarantee is secured that the work will be substantially carried out, than would result if European methods were pursued. The concrete quarters are built after the following specification.

SPECIFICATION.

Centering and Aligning.—The centering can be cheaply and expeditiously made by simply raising a mound of earth the whole length, and in contour, of the inside section of the building, in rammed layers, and having the outer surface dressed off with clay. After construction the earth is easily removed through the doorways. The outer surface of the concrete is supported either by a clay wall 1 foot thick, banked with earth, or by a framework of rails and sleepers or boards, Plate 10, Figs. 1, attached, and, as is preferable, and as cheap, where several blocks are to be erected, the centering can also be put up of the same.

In erecting the centering, it will be found useful in setting out the work, and for purposes of check afterwards, to put up light poles with cross-piece, from which to stretch aligning cords to pegs in the ground; this at either end of the block. A light template is required for the curved roof, Plate 10, Figs. 2.

Method of Construction.—1. After the concrete foundations have been rammed solid, the centering and outer framework should be erected, and the door-frames fixed in position. The frames to have projecting pieces 6 inches down into the concrete, and 3 inches upwards to fit into sockets in the stone lintels. They should also have temporary flanges, Plate 10, Figs. 3, clamped on either side, to retain the concrete walls during construction. The stone lintels should be as light as possible, say 1½ inch thick, and not overhang more than 6 inches, being laid nearly horizontal across, and bedded 12 inches in the concrete at either end. The lintels should be kept ½ inch clear of the top of the door-frame to allow for settlement, and be supported till the concrete sets. The lintels can be plastered with a slope on the top to shed rain.

2. The concrete should be laid in a 6-inch layer, and well consolidated with wooden hand-tappers; the top surface should then be slightly picked up, preparatory to laying the next course, and so on. The tie-rods, fitted with their large washers, 6 inches by 6 inches by ½ inch, and cotters, or roughly forged heads, are laid in position about 1 foot above the springing of the arch, the concrete being carefully packed against the washers. The walls are given a slight batter to meet the thrust of the arch, so that tie-rods are not necessarily required, but are simply intended to lend additional stability to the structure.

3. The concrete in the arch should also be laid in 6-inch courses, and carefully beaten in the direction of the thrust, and with a face parallel to the radius; a movable board with slightly curved face,

and which can be adjusted as required, can be used to retain the outer surface while being consolidated. Usually the centering, &c., can be removed in about fifteen days after completion of the concrete, the outer surface smoothly, and the inner roughly, plastered, and the partition-walls dividing off the units erected of sunburnt brick, or country-burnt brick set in clay mortar. The interior can then be whitewashed. This method of putting in the partition-walls last, and of such material, is found to save the concrete from cracking over the walls, as it usually does on other works where the walls are erected first, expansion or contraction being thus interfered with. Another advantage is, that the centering can be more easily placed as well as removed.

Concrete.—Broken stone, hard brick or kunkur cubes, not exceeding 1½ inch in foundations, and ¾ inch in superstructure.

Mortar, composed of 2½ parts of sand or soorki to 1 part of lime, thoroughly ground for say six hours, in a bullock-power mill.

The above to be well worked up together in the proportion of 1 part of mortar to 2 parts of metal.

ESTIMATED COST per UNIT.

	£.	s.
Plate-layers' quarters[1]	5	5
Menials' (A class) quarters	6	15
Gate Lodge	6	0
B class clerks' quarters	14	5
C „ „ „ (Plate 10, Figs. 4, 5, 6, 7)	24	5

The Paper is accompanied by several drawings, from which Plate 10 has been engraved.

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[1] These are the same as class A, without courtyards.

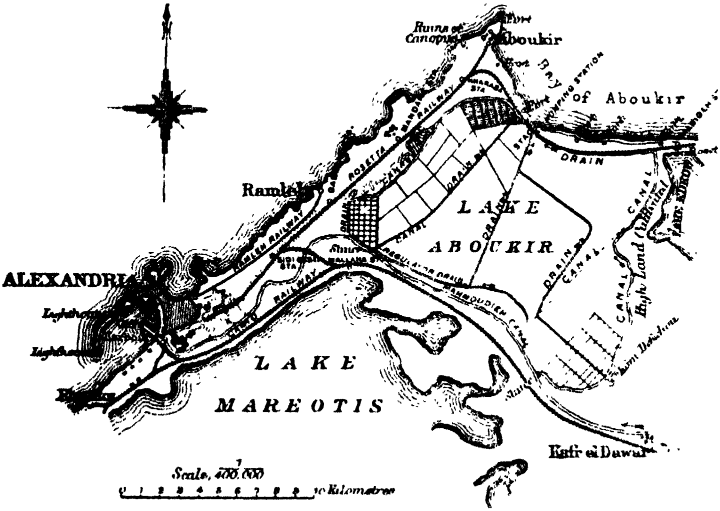
(*Paper No. 2444.*)

"The Reclamation of Lake Aboukir."

By Herbert Gurney Sheppard, Assoc. M. Inst. C.E.

With the scanty means of information available on the subject of the reclamation of salt lakes, it may be of interest to relate the methods adopted on the above works. In every country there are local circumstances always existing, which necessitate special means to effect the object desired; and in no country is this more so than in Egypt—a land unique in every way, both in its original formation and in its subsequent development as an agricultural country.

Fig. 1.



Lake Aboukir lies 6 miles to the east of the City of Alexandria (*Fig. 1*), and forms one link of the chain of salt lakes which extend all along the northern coast of Lower Egypt from Alexandria to Port Said. This immense area, if brought under cultivation, would largely augment the revenue of the country. Unfortunately, the beds of these lakes lie some feet below the level of the sea, and can only be drained by means of pumping-engines. This entails a heavy charge on lands so reclaimed, and they must give promise of ultimate high value, before such expense can be reasonably incurred. Such promise the reclamation of Lake Aboukir was considered to hold out. On the outskirts of a rapidly growing city of two hundred and fifty thousand inhabitants, and with two railways, and a large navigable canal encircling three of its sides, it possessed advantages from a commercial point of view shared by none of the other lakes.

Another fact in its favour was, that a considerable portion of it had undoubtedly been under cultivation in olden times, as evidenced by numerous remains of villages, stone walls, earthen vessels, coins, and a variety of other articles found in the bed of the lake. With lands in the vicinity of Alexandria commanding an average annual rent of £5 per acre, it was considered that the reclamation of Lake Aboukir might prove a profitable undertaking, and Mr. William Grant having obtained a concession with that object from the Egyptian Government, an English Company was formed to carry out the necessary works.

The concession was dated March 12th, 1887, and all the main works had to be completed by the 31st of December, 1890.

This allowed little time for the preparation of the necessary surveys and plans, and after a report upon the project had been drawn up by Mr. James Abernethy, Past President, as consulting Engineer to the Company, the works were commenced in May 1887.

THE LAKE.

The area of the lake, included in the above concession, is 29,621 feddans, or 30,717 acres (1 feddan = 1·037 acre). In section it is saucer shaped, flat in the centre, and gently rising towards the edges. The average level of the centre portion is about 1 metre (3·28 feet) below mean sea-level; and none of the land, even around the edges, is high enough to drain into the sea, which is kept out by means of a stone sea-wall. Inside of this latter is an embankment carrying the railway to Rosetta, and this practically forms the boundary of the lake for 10 kilometres (6·2 miles) on the north side. Within this distance there are three stations. The lake lands are thus well protected from any possible encroachment of the sea. The same railway again skirts the edge of the lake at two points along its north-western shore, both provided with stations, and the Alexandrian and Ramleh Railway runs within a mile of its extreme western point. The south-west and south sides, for a distance of 14 kilometres (8·7 miles), are bounded by the Mahmoudieh Canal, although the canal actually touches the lake at only two points within this length. It is, however, so close as always to serve as a cheap means of transit to and from the Port of Alexandria. During the dry season it is supplied with water from the Nile by large pumps at Atfeh, so that boat traffic is constantly maintained.

The eastern shores of the lake gradually merge into somewhat higher land, separating it from Lake Edkou; the soil of this elevated ridge is exactly similar to that of the bed of the lake; but in consequence of its superior drainage, it has already been most successfully cultivated, and villages abound from one end of it to the other.

To be strictly accurate, Aboukir was not a lake at all, because during the hot weather, from May to August, it used completely to dry up. During the high Nile and winter months, the water collected to about 9 inches in depth, over the bed of the lake, and there it remained until evaporated in the following summer. This water was partly due to rain, and partly to drainage water from coterminous cultivated lands. There is little or no infiltration from the sea.

Before the present works were taken in hand, Aboukir was merely a huge salt-evaporating basin. During summer, the salt would lie in the depressions in sheets of dazzling whiteness, 3 or 4 inches thick, much resembling snowy ice in colder regions. The salt was nearly pure chloride of sodium, and the sale of this being a government monopoly, it was carefully guarded by a large squad of watchmen in the pay of the coastguard service, distributed round the borders of the lake.

The presence of this large amount of salt may be variously accounted for: 1. It is inherent in the whole of the deltaic formation of Lower Egypt; 2. The sea, standing at a considerably higher level, would tend to force any salts to the surface, even if the salt water itself could not penetrate to an appreciable extent.

3. According to historical records, a great storm in the year 1715 breached the sea-wall above referred to, and for many years the lake remained open to the sea.

Year by year, then, has seen this salt probably growing in quantity, alternately dissolved by the winter rains and dried by the summer heat. It was this, and this only, that put an end to the successful cultivation of the bed of Lake Aboukir. The methods adopted in getting rid of the salt will now be described.

THE PUMPING STATION.

Two ways present themselves for draining Lake Aboukir; one by raising the water by large pumps and discharging it into the sea; the other by means of siphons or culverts under the Mahmoudieh Canal, and running off the water into Lake Mareotis, the mean level of the water of which is about 1½ metre (4·92 feet) below the bed of Lake Aboukir.

The latter would naturally commend itself as the cheaper and more efficient plan; but, unfortunately, it did not meet with the approval of the Egyptian Ministry; and pumps were specified as a *sine quâ non* in the concession. There is therefore no occasion to describe the merits of the other method of drainage.

A convenient site, then, having been found for the pumps on the line of railway to Rosetta, and within 350 metres (383 yards) of the sea, the pumping-station was erected.

As the only fresh-water available had to be conveyed in tanks by railway from Alexandria, a distance of 14 miles, it was decided to build all the foundations both of engines and buildings in cement, using salt-water only. The concrete for the foundations consisted of broken pottery 6 parts, sand 3 parts, and Portland cement 1 part. The broken pottery, if clean and well broken, forms an excellent substitute for stone, and is found in large quantities by excavating the ancient sites of villages or kôms. Upon the concrete the engine foundations 9 feet 6 inches in depth were built of brick in cement, the engine-room floor being on a level with the rails of the Rosetta line.

The engine and boiler-houses were of corrugated iron. The engine and pumps, by Messrs. John and Henry Gwynne, of Hammersmith, consisted of two "Invincible" centrifugal pumps with suction-pipes 48 inches in diameter, each worked by a separate horizontal direct-acting compound surface-condensing engine, with cylinders 17 inches, and 32 inches in diameter, and stroke of 27 inches. The circulating water was pumped by means of two independent 6-inch "Invincible" pumps. Four Galloway boilers 7 feet in diameter, and 20 feet long, supplied steam to the engines; two only are used in daily practice. The diameter of the pump-fan is 6 feet 9 inches, that of the pump-casing being 15 feet 6 inches outside measurement; the latter is an excellent casting in one piece, and weighs 13 tons. Great difficulty was experienced in transporting these large castings by rail from Alexandria, as their dimensions did not allow of trains passing each other, and special precautions had to be taken to keep the lines clear. These engines were specified by the makers, each to discharge 175 cubic metres (228·9 cubic yards) of water per minute with a lift of 11 feet, and a consumption of 2 kilograms (4·41 lbs.) of Welsh coal per actual HP. per hour in water raised. That they were well within their contract quantity the following figures will show :—

Result of Pump-Trials, 23rd and 24th October, 1888.

		Right-hand Engine.	Left-hand Engine.
Duration of trial	hour	1	1
Mean boiler-pressure . . .	lbs. per square inch	80	90
„ vacuum	inches	27½	27
„ revolutions per minute of engine and pump		80	80
„ lift	feet	9·10	8·87
Discharge over sill per minute . .	cubic metres	174·41	195·0
Density of intake water	per cent.	3·9	3·9
Effective work in water raised, or W.HP.. . . .		110·01	119·92
Indicated HP.	hp. cylinder	106·72	114·64
„	lp. „	91·68	82·98
Total indicated HP.		198·40	197·54
Efficiency of pump, or $\frac{\text{W.HP.}}{\text{I.HP.}}$		0·55	0·60
Consumption of Cardiff coal—			
Per hour	lbs.	412½	462
„ I.HP. per hour	„	2·07	2·33
„ W.H.P „	„	3·74	3·85
„ „ „	kilo.	1·70	1·75

The discharge was carefully measured over a wooden weir built across the mouth of the outfall; this latter was 350 metres (383 yards) in length, by 37 metres (40·5 yards) in width, and, cutting through the sea-wall previously mentioned, delivered directly into the sea. The weir was 33 feet long with a clear overfall; but the shifting sandy bottom rendered it extremely difficult to keep it watertight for any length of time; and excessive leakage, very difficult to estimate, put an end to further experiments under varying lifts and boiler-pressures.

It will be noticed that, in the particular trial given, the right-hand engine was only working with a boiler-pressure of 80 lbs. per square inch, and, at 80 revolutions per minute, was discharging 174·41 cubic metres (228·13 cubic yards). In a subsequent trial on the same day, but under a boiler-pressure of 90 lbs., this engine ran at 83 revolutions with the same lift, and discharged 180·7 cubic metres (236·36 cubic yards) per minute. The indicator diagrams not being satisfactory on this occasion, the results are not tabulated.

The full lift of 11 feet, required by the makers to ensure the minimum consumption of fuel per water HP., could not be obtained, as there was at the time too much water in the drains. With a higher lift the efficiency of the pumps is much improved. Only two boilers were in use during the trials, and, without forcing, supplied all the steam necessary. Although at times nearly 400 tons of water pass every minute through these pumps, there is no sign of swirling or vortex action, either on the intake or on the outfall sides. The suction-pipes are bell-mouthed, the water being trained up to them by masonry walls placed at suitable angles. The discharge-pipes, of cast-iron, gradually increase in diameter from 4 feet at the fan to 8 feet at the outfall. The level of the water in the outfall, which is sea-level and never varies more than 18 inches, averages about 2½ feet above the tops of the pipes. The very large diameter of these pipes reduces the velocity of efflux at the mouth to only 2·04 feet per second, with a discharge of 175 cubic metres (228·9 cubic yards) per minute for each pump.

The economy of these engines in daily practice is shown by the average consumption of coal, of very indifferent quality, during the six months from January to June 1889.

According to the engine log-book, this was only 9·20 cwt. per hour for both engines, including lighting up, working with an average lift of only 9·10 feet, and an average speed of 75 revolutions per minute. Now, assuming an average discharge, at this low lift, for both pumps, of 350 cubic metres (457·8 cubic yards) per minute, this will give a coal consumption per water HP. (219) per hour of 4·7 lbs., or 2·13 kilograms, the average density of the water pumped being 1·03. With low lifts, a difficulty is found in keeping up the speed of the engines; with higher lifts comes increase of speed, and consequently greater discharge, the engines being designed for a normal lift of 11 feet, which, however, is seldom obtained.

In calculating the duty of the pumps, the density of the drainage water forms an important item, and is daily taken at the pumping-station by a delicate hydrometer. Thus it can be easily ascertained if the water used in washing any part of the lake is being turned into the drains before it has dissolved its due proportion of salts.

The average density of the drainage water at the intake is about 1·03, but during the month of April, 1888, when the lake was being first dried, and the water was low in the drains, it was as much as 1·12. When the pumps are discharging 350 cubic metres (457·8 cubic yards) per minute, this is equivalent to nearly 42 tons of soluble salts extracted from the soil per minute. Although this, at first sight, appears to be a very large figure, it is insignificant when compared with the total amount of soluble salts contained in the lower lands. From analysis, given further on, this exceeded 10 per cent., and the weight of the soil dried being 141 lbs. per cubic foot, to effectually sweeten it to a depth of ½ metre (1·64 foot), 466½ tons of salts had to be extracted from each feddan of land constituting the bed of the lake. Such an absolute freedom from salts is, however, unnecessary for many crops, the best of soils in these parts often containing 2 per cent. of soluble salts. The Author found that the percentage of chlorides in the water of the Mahmoudieh Canal varied from 0·146 in August and September (during flood) to 0·936 in July, when the Nile is at its lowest. During the first pumping season, and before any of the fresh-water canals were made, the boilers were supplied by tanks of fresh water brought by rail from Alexandria, the water in the intake being too salt to use with safety. The cost per month of working these pumps, with day shift only, is—

	£.	s.	d.
Staff	45	10	0
Coal, 165–6 tons at 23s.	190	8	9
Stores	15	0	0
	£250	18	9

Therefore, with the pumps discharging 350 cubic metres (457·8 cubic yards) per minute, with a lift of 9·1 feet, and coal consumption of 9·2 cwt. per hour as before, but working twelve hours per day, the cost per 1,000,000 cubic metres (1,308,000 cubic yards) of water lifted is £33·1.

The total cost of the pumping-station was £19,962, which, under a normal lift and discharge, is equivalent to £77 6s. per water HP. The need of using Portland cement in the whole of the masonry, where lime would have served equally well, had fresh water been available, affected the cost considerably, as also did the heavy custom dues levied on all the machinery imported.

THE CANALIZATION.

The whole of the water, both for washing and for cultivation, is obtained from the Mahmoudieh Canal, by means of two head-sluices, each with two openings of 1·50 metre (4·92 feet) by 1·25 metre (4·10 feet), capable of discharging about 500 cubic feet, or say 14 cubic metres per second, with a maximum head of 2·8 metres (9·2 feet). The general section of the lake being saucer-shaped, as before stated, the Author determined upon two systems of canals for each half of the lake; a high-service canal running round the boundaries of the property, and a low-service canal across the centre. The two, although supplied by the same head-sluice, are separated by a regulator, which, by means of loose planks sliding in a groove, regulates the difference of level amounting to about 1 metre (3·28 feet). Water is thus supplied by gravitation to all parts of the lake, and, by a network of distributory canals, was made to conform to varying levels. The greatest head of water held up in the main canals did not exceed 1·3 metre (4·26 feet), the banks being 2·25 metres (7·37 feet) wide on the top, and with slopes of 1½ to 1. As a rule, however, the slopes were only 1 to 1, both for banks and cuttings, and were found to stand well, the clay soil of the lake-bed forming an excellent puddle. Where possible, the main canals were designed with a surface-slope of 7 centimetres to the kilometer $= \frac{1}{14{,}285}$; the bed-width varied from 7 metres (23 feet) to 2 metres (6·56 feet), the discharge being calculated on the basis of 30 cubic metres (39·24 cubic yards) per feddan per day; 24 cubic metres (31·39 cubic yards) is the average quantity of water required by Egyptian crops, and 40 per cent. of this will find its way into the drains. The length of the high-service boundary canal supplying the southern half of the lake is 24½ kilometres (15¼ miles), that for the northern 12 kilometres (7½ miles). Three large main drains divided the whole lake into four parts, of about 7,000 feddans each; they were given a bed-slope of 5 centimetres to the kilometre $= \frac{1}{20{,}000}$, which was all the very flat nature of the land would allow, even this necessitating a depth of cutting at the pump end of 2¼ metres (7·38 feet). The bed-width varied from 3 metres (9·84 feet) at the head to 8 metres (26·24 feet) near the intake. The drains are made considerably larger than theory requires, in order that they may act as a collecting-basin for the pumps. These, as a rule, work only during the day, and to keep down the level of the water at the head of the drains, distant, perhaps, 15 kilometres

($9\frac{1}{3}$ miles) from the pumps, it is imperative that the flow be maintained throughout the night.

Besides the above main drains, a large drain was cut outside of the high-service boundary canals, and this extended nearly three parts round the lake, wherever there was cultivated land outside the borders. Its use was to tap all the small drains, which, before Lake Aboukir was drained, discharged into that depression. This water, having been thus collected and brought under control, was utilized for washing certain low parts of the lake containing an undue percentage of salts. The works having been pushed forward with great energy during the seasons of 1887 and 1888, the main system of canalization was completed by December 1888. These, together with the pumping-station, represented the requirements of the Egyptian Government, according to the deed of concession, and, having been duly examined and reported on by a special commission of Government engineers, the terms of the contract were stated to have been duly fulfilled, and in March 1889, or one year and nine months from first breaking ground, the whole area of 29,621 feddans became the property for ever of the present company. Having obtained possession of the land, the next point was to develop it by eliminating the salts, and by means of the necessary roads, drains, and distributing canals, to put it in a state capable of the highest degree of cultivation.

THE WASHING OF THE SOIL.

In anticipation of ultimate possession, much of this smaller work had been executed early in the season of 1888, with a view to utilizing for washing purposes the flood-waters of that Nile, usually available from August to December. The Nile, however, failing to rise properly during this year, washing operations were not commenced till the 14th of September, and then only on a small scale. In Egypt three systems of reclaiming salt-lands are practised.

1st. By "colmatage," or, as it is called in England, "warping." During the time of high Nile, water heavily charged with silt is run over the land to be reclaimed, and after this is deposited the clear liquid is drawn off, and the process repeated. In course of time a sufficient depth of silt is obtained to produce a crop, the drains preventing the salts from rising and contaminating the new soil.

2nd. By washing up the salt. This is effected without the intervention of drains, and merely by passing fresh water continuously over the surface of the soil to be sweetened.

3rd. By washing down the salts. This is done by dividing the land into small plots by drains and dykes, the size of the plots depending upon the saltness and quality of the soil; the spoil from the drain-cuttings forms small banks round the plots, and enable them to be kept flooded constantly with fresh water. The action is thus one of forcing the salts down into the drains by mere pressure, due to the weight of water, and assisted no doubt by a process of diffusion.

The first plan can only be adopted to advantage on lands situated near the Nile, or within reach of canals having a silt-carrying velocity. The water of the Mahmoudieh Canal has deposited most of its silt before arriving at Lake Aboukir, and is useless for this purpose.

The second method of washing given is slow, extravagant in the quantity of water required, and therefore not practicable in cases where it has to be pumped off. When ordinary waste drainage-water, containing, perhaps, only a trace of salt, is allowed to pass over very salt lands, the latter will be found in time to become practically sweet, and generally covered with a thick crop of reeds, rushes, and rank grass. There are several examples of this process of natural reclamation in Lake Aboukir, at places where the drainage water from outside lands discharges into the lake. In one instance nearly 2,000 feddans of land had almost entirely been deprived of its salt by this means, and only required to be properly drained and irrigated to exchange its wild vegetation for useful crops of clover and barley. An attempt was made to imitate this method of washing on 6,000 feddans of bare salt-lands on the northern side of the lake, and for this purpose it was divided into basins of about 500 feddans each by heavy banks. It was proposed to keep these basins always flooded by fresh water from the boundary canal already mentioned, drawing it off from time to time, by means of the pumps, as soon as it had absorbed a due proportion of salt. This, in fact, was the original plan for reclaiming the whole of the bed of the lakes, and was only abandoned when it was found that the wave-action, due to wind over such a large surface of water, eroded and breached the banks to an extent quite out of proportion to their utility. It was also found that the water did not penetrate the stiff clay soil, and that after the first washing or two, the water had to remain many days before it reached a density which justified the expense of pumping it off. The plan then ultimately adopted, or some modification of the same, is shown by *Fig. 2*, which represents the method of dividing up a "hoshay," or plot, of about 70 feddans area.

The land is first divided into rectangular fields, A, B, C, D, 1,000 metres (1,094 yards) long by 300 metres (328 yards) broad. It is then ploughed up, either by steam or bullock. To reduce as much as possible the effect of wind action upon the washing water, and to secure better drainage, the "Hoshay" is subdivided into twenty smaller plots, 150 metres (164 yards) long by 100 metres (109 yards) broad. With continuous changes of water, the washing out of the salt is now taken in hand, and is carried on continuously, until the soil is considered sweet enough to grow a crop of rice, or "dineba"; the latter is a species of millet (*Panicum crus-galli*) and occurs plentifully in the rice crop. It forms a capital green fodder for cattle, and thrives on salt-land much better than rice.

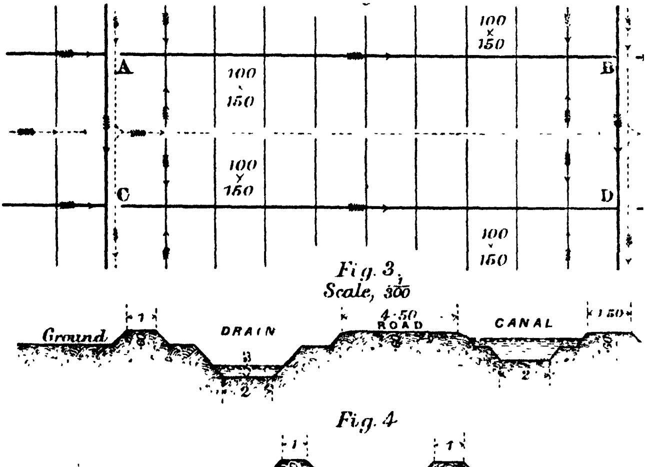


Fig. 3,
Scale, $\frac{1}{300}$

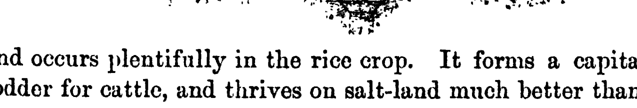
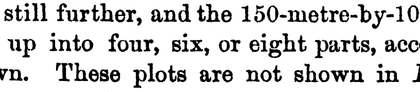


Fig. 4



Before, however, any crop can be sown, it is usual to carry out the reticulation still further, and the 150-metre-by-100-metre plots are again split up into four, six, or eight parts, according to the crop to be grown. These plots are not shown in *Fig. 2*, as the work is usually executed by the cultivators themselves. Referring to *Fig. 2*, the thick lines represent the larger drains, in communication with the main drains leading to the pumps; parallel with these are the distributory canals for fresh water, shown in dotted lines. The spoil from these two cuttings goes to form the intermediate roadway (*Fig. 3*), 4½ metres (14·76 feet) wide; the section through these parallels A C, or B D, is shown in *Fig. 3*, and *Fig. 4* is a section through the collecting drain A B, or C D. The small cross drains shown in *Fig. 2* as separating the 150-metre-by-100-metre plots are usually not more than 0·70 metre (2·30 feet) deep, and 0·25 metre (0·82 foot) bottom width, but they vary with the nature of the soil. The arrows represent the direction of flow of the drainage or irrigation water; the latter is distinguished from the former by being dotted.

The following analyses of the soil of Lake Aboukir, by Dr. Voelcker of London, will show that there was a most unusual quantity of chloride of sodium to be dealt with, the extraction of which would require large quantities of water applied in a systematic manner:—

ABOUKIR SOIL ANALYSES. Dried at 212° Fahrenheit.

——	No. 1.	No. 2.	No. 3.	No. 4.
Oxide of iron	11·69	11·04	3·50	11·71
Iron pyrites	0·08	0·11	0·12	0·10
Alumina	6·36	10·88	4·54	11·95
Lime	2·08	7·73	29·52	8·03
Magnesia	1·79	0·93	1·10	0·50
Soda	0·79	..	0·49	0·41
Chloride of sodium	8·11	8·56	1·62	0·01
Potash	0·65	1·23	0·43	0·67
Sulphuric acid	2·23	2·56	6·16	0·14
Carbonic ,, ...	0·19	4·75	20·15	5·59
Phosphoric ,, ...	0·16	0·19	0·35	0·38
Insoluble silicates and sand .	62·23	45·81	31·51	54·27
*Organic matter	3·64	6·21	0·51	6·24
	100·00	100·00	100·00	100·00
*Containing nitrogen . . .	0·035	0·070	0·035	0·096
= Ammonia	0·042	0·079	0·042	0·116

Samples No. 1 and No. 2 were taken from the bed of the lake, and represent the average quality of the soil over an area of, say, 25,000 Feddans. It consists, for the depth of a metre (2·46 feet), of a stiff loamy clay, of 2·25 specific gravity, black in colour, and of extreme fineness. Below this there is generally a sandy stratum of varying thickness, forming very efficient subsoil drainage;

sample No. 3 is somewhat lighter, with much lime, and was taken from some of the higher land lying about 1 metre (3·28 feet) above the average level of the lake. Having been washed by many winter rains, it has nearly parted with its salt, and, in fact, has already produced a scrubby winter crop of barley, planted in past years by itinerant Bedouins.

Sample No. 4 is taken from some already fully cultivated land in the vicinity of the lake, and is for comparison only; it produces average crops of cotton, maize, and other cereals, and is worth about £5 a year rental. From the above analysis it will be seen that the soil of Lake Aboukir is essentially alluvial, and formed by the deposition of Nile silt in a similar manner to all the rest of the Delta. It possesses all the ingredients of a first-class cultivable land, and only requires, by means of artificial drainage and washing, the elimination of its superabundant salts of sodium and magnesia to become so.

On examining the analysis, it will be noticed that chloride of sodium forms about 50 per cent. of the soluble salts; but in an analysis, by Dr. Voelcker, of a sample of salts contained in the drainage water from the land being washed, the proportion of chloride of sodium is found to have increased to 70 per cent., magnesia and sulphate of lime being also present. This confirms, in a very marked degree, the fact well known to every agricultural chemist, that soils have the power of retaining their valuable chemical constituents, whilst readily parting with those either noxious or of little importance, and there need therefore be no fear of excessive washing deteriorating the soil in any way.

To ascertain the progress of eliminating the salts from the soil, experiments are made at frequent intervals in the Company's laboratory, which is especially arranged for the purpose.

It was found that the well-known nitrate-of-silver-test for chlorides was the most accurate and easily managed, and was used exclusively where chlorides were concerned. It may, or it may not, be correct to assume that chlorides are the only, or the main noxious ingredient likely to be present in the soil; but the broad fact remains that when the chemical tests show absence o' chlorides in excess, the land invariably follows with a growth of crop in direct proportion to the decrease of chlorides.

In Dr. Voelcker's analysis, in soils taken from the bed of the lake, the chlorides amounted to 8 per cent. After one year's continuous washing, this was found to have diminished on an average to about 2 per cent. at a depth of 1 foot below the surface, although in many cases, where the washing had been done under more favourable conditions, only ½ per cent. or less of chlorides was found.

With 2 per cent. of salt in the soil, a fair crop of dineba, 2 feet high, can be grown; with 1 per cent. it attains its full height of 4 feet, and sells as a standing crop at from 20*s.* to 25*s.* per acre. For "berseem," or clover, the percentage of salt should not exceed ½, and about the same for "sabaini rice."

It must be pointed out here, that the Administration does not itself seek to cultivate land known to be capable of bearing a good crop, but only such semi-reclaimed land requiring special treatment, with a view more to its future value than to any profits to be made on what may be called "reclaiming crops." It would therefore be useless to quote any figures relating to the cost of raising any particular crop, because, as soon as any land has been proved sweet, it passes out of the hands of the Administration, and is let to the "Fellah."

THE COST OF WORKS.

The total amount of earthwork executed from May 1887 to September 1889 was 1,457,597 cubic metres (1,906,537 cubic yards), representing the excavation of about 72 miles of main canals and drains, some 25 miles of minor drains, and the preparation for washing and cultivating of 4,140 feddans, as shown in *Fig. 2.* The whole work was performed by contract in the ordinary native fashion with basket and hoe. The rate paid for the larger work was from 2 to 2¼ piastres tariff per cubic metre, say 4½*d.* per cubic yard, which included dressing the banks and keeping the works free of water.

For such work, as shown in *Figs. 3* and *4*, ¾ piastres tariff per cubic metre, or 1·4*d.* per cubic yard was the usual price paid. The total expenditure in Egypt for the whole works amounted to £77,977 5*s.* 4*d.* up to the end of September 1889. Besides this, there was the cost of financing the project in London, with which the Author was not concerned.

Taking the above sum, then, as the actual amount spent in the reclamation of the lake as far as it has gone, the cost per acre and feddan for the three stages of the work of reclamation will be as follows:—

	Per Feddan.	Per Acre.
	P.T.	£.
1. First stage. General expenses £. E. 72,586. Main earthworks, pumps, &c.	245·9	2·43
2. Second stage. Division into basins of 500 feddanu area	28·8	0·28
3. Third stage. Subdivision into small fields, as per *Fig. 2*	76·5	0·75
4. Labour and washing for one year	27·0	0·26
5. Steam cultivation	17·0	0·16

Omitting item 2, which, as already explained, is an unsatisfactory system, the total cost of the works was at the rate of 366·4 piastres tariff per feddan, or £3 12*s.* per acre exclusive of any charge for pumping drainage water. This cannot be correctly estimated, as no separate record of daily discharges of drains from lands undergoing washing treatment has been kept. The above cost, then, can only be considered as approximate, some land, parting readily with its salt in a few months, while others of a heavier nature will require more than a year of continuous washing. It is the object of the Administration only to carry out as much work as will ensure the letting of the land to the fellaheen who will not risk even their labour and seed in what might prove to be an unproductive soil; it is imperative, therefore, that the Administration should cultivate to a limited extent such reclaiming crops as "dineba," rice and clover, in order to show the capabilities of the soil when once deprived of its salt. The cost of such cultivation, being necessarily a revenue charge, is not included in the above cost of works per feddan. Of course it will be some years before the land acquires its maximum value, and the present rental is usually, therefore, £1 per feddan for the first year, £2 for the second, and £3 for the third. These prices are readily obtainable where a crop of "dineba" shows the land to be no longer impregnated with an excess of salt, and are sometimes exceeded, where the soil has been naturally washed and drained for many years. After the third year, when the land has been worked into good condition, and the country settled, it will, without doubt, let as readily at £5 per feddan as do similar lands in the vicinity. Villages are erected at the cost of the Administration on suitable sites for the use of the tenants; they consist of the usual mud huts, 3½ metres (11·48 feet) square, with dome roof; built merely of sun-dried bricks, and plastered over with clay and chopped straw; they cost about £3 10*s.* per house.

The steam cultivation is carried on with a pair of Fowler's 8-HP. ploughing engines, using a nine-tined cultivator or grubber. Where the land is free from long reeds, the engines can work up 13 feddans per day, at the cost above stated, namely, 17 piastres per feddan, the consumption of coal being under 1 ton per day, and costing at the site 30*s*. per ton. Where there is much rank grass, a Cuban plough is used; the natives, however, prefer the work of the cultivator as being more like their own primitive implement.

The Author was engaged as Resident Engineer on the works, acting under the advice of Mr. James Abernethy, Past President, as Consulting Engineer.

The agricultural problem involved in the transforming of this barren tract into rich, cultivated land, is now being worked out by Mr. V. Lang-Anderson, as manager of the estate in Egypt; and it is due to his kind assistance that the Author was able to collect much of the information contained in the above notes.

The Paper is accompanied by a map and two diagrams, from which the *Figs.* in the text have been engraved.

(*Paper No. 2417.*)

"On the Action of Quicksands."

By WILFRID AIRY, B.A., M. Inst. C.E.

THE peculiar property of quicksands in swallowing up bodies which may happen to come upon them is well known. Articles great and small, which would be perfectly well able to float in water, are unable to do so in a quicksand, although it is of greater specific gravity than water, and are gradually drawn down. Vessels which have been stranded on quicksands by a falling tide have been unable to float off when the tide has again risen, and after being swamped have settled down deeply into the sand; and, in the descriptive accounts of such accidents, it is often stated that there appeared to be something mysterious in the way the vessels disappeared. The Author has thought that it might be useful to offer an explanation of these phenomena, and to show by experiment the effect of such quicksands.

When a vessel or other body floats in water, it floats by virtue of the vertical components of the water-pressures around it. If, for instance, the body be a rectangular box floating with its sides vertical, then the box floats solely by reason of the water-pressure on its bottom; and, if its bottom were protected in any way from the action of the water, the box would not float. This is precisely what happens in the case of a quicksand. The mingled mass of water, silt and sand which constitutes a quicksand forms a semi-fluid which is not capable of transmitting the full hydraulic pressure of the water. When therefore a vessel comes upon such a quicksand her bottom is partially protected from the action of the water, and she is unable for a time to float up, even after the water has risen considerably higher than the ordinary flotation-level of the vessel. Sooner or later she will float up if she be not swamped, as will be seen in the course of these investigations.

A quicksand may be defined as a mass consisting chiefly of sand; but partially also of silt or argillaceous matter so intimately mixed with water, that it forms a semi-fluid, having all the properties of a fluid, but in a minor degree. The most familiar instance of a quicksand is that of the sand on a beach, where it is kept alive and mixed up with water, soil, and organic matter by the action of land-springs rising from below it; this often forms a very dangerous quicksand. But the admixture with water may be due to other causes. In general, the tendency of silty sand is to sink together and form a firm, compact body, having apparently none of the properties of a quicksand; but this very sand, when its surface is disturbed by mechanical violence, such as the force of the waves, will, to a certain depth, become a quicksand. The grains of sand become so intimately mixed up with the water that the mingled mass of sand, silt, and water, becomes a semi-fluid covering the solid body of the sand to a certain depth.

It will be readily seen that the extent to which a mass of sand is capable of becoming a quicksand will depend upon the fineness of the sand, and the extent to which it is mixed with silt or argillaceous matter. If the sand be sharp and clean, no matter whether it be coarse or fine, it has very little of the nature of a quicksand; but if it be in a fine state of division, and be mixed with silt, &c., the particles will be readily held in suspension by the water, and it may form a deep quicksand. If a vessel be left upon such a quicksand by the falling tide it will sink considerably into it, and the sand will form a very complete and sufficient seal round the bottom; and on the subsequent rise of the tide the vessel will not float when the water has reached the ordinary flotation level, but a considerable time will elapse before the water can communicate sufficient of its pressure through the seal of sand to float up the vessel; and in the meantime the water will have risen considerably higher than the ordinary flotation level of the vessel.

In order to verify these conclusions, and to obtain some reliable measurement of the effect of such quicksands, the Author made experiments in the following manner. A trough was prepared containing several inches in depth of the sand to be experimented upon, with water above it; and the surface of this sand was worked up by hand so as to become "quick" to a small depth. A rectangular watertight box, 12 inches long by 8 inches broad, was then floated in the water, and was forced down upon the sand with its sides vertical, and sunk into it to the depth of ½ inch. When released from the pressure by which it had been forced down it remained sunk, and was held down by a great force, due mainly to the atmospheric pressure, while at the same time its bottom was protected from the action of the still greater external pressures by the seal of the sand. The time was then observed which elapsed before the external pressures were able to break through the seal of the sand sufficiently to float up the box.

The general statement of the problem is given on the following page.

In the annexed diagrams, *Fig. 1* represents the box when floating. X X is the surface of the water, and C C the surface of the sand. B X is the draught of water of the box; *Fig. 2* represents the same box when forced down upon the sand and sunk into it a small depth B C. K X in *Fig. 2* is equal to B X in *Fig. 1.* The box is assumed to be rectangular and its sides in all cases vertical.

The pressure per square inch on the bottom of the box due to its own weight is that of a column of water B X (*Fig. 1*), and the pressure of the atmosphere is that of a column of water about 34 feet high; also the pressure on the surface of the sand due to the weight of the water above it is that of a column of water C X

Fig. 1.

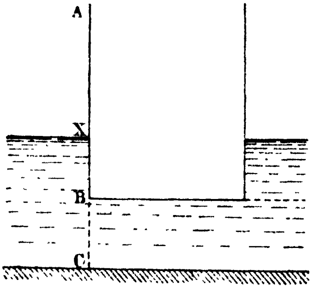
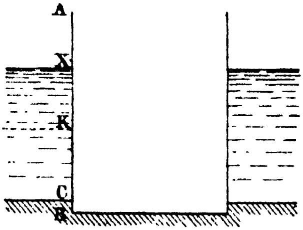


Fig. 2.



When therefore the box is sunk as in *Fig. 2*, it is retained in that position by its own weight added to the pressure of the atmosphere (B X + 34 feet), and is partially protected from the immediate action of the external pressures tending to force it up (C X + 34 feet) by the seal and resistance of the sand round its bottom. A portion of the external pressure is transmitted through the mass of the sand directly to the bottom of the box. The remaining portion of the external pressure will proceed at once to attack the seal of the sand round the bottom of the box, and will endeavour to communicate its own pressure to the bottom of the box through the sand lying immediately around it. This process is very gradual, but perfectly certain, and when, by the joint action of the two portions of the external pressure, the downward pressure of the box upon the sand (B X + 34 feet) is overcome, the box will float up. The problem is to find by experiment what length of time will elapse before the external forces have overcome the resistance and seal of the sand sufficiently to float up the box.

Many experiments were made with different kinds of sand, both fine and coarse, with the general result that clean and sharp sands, such as fine blown sand from the Norfolk coast, clean beach sand, washed Thames sand, &c., had few of the properties of a quicksand. But the case was different with sands which, although apparently clean, contained an admixture of clayey matter. When the box was forced down upon such sands it felt quite free as if floating, and could be moved about laterally with great ease; but it offered very great resistance to a vertical pull. The sands for which the average results are quoted in the following Tables were:—(1) Fine sand from the Blackheath Tunnel; (2) Sand from the Woolwich pits, such as is used for foundry castings. These are taken as representing the properties of quicksands containing a moderate admixture of clayey matter. With a larger proportion of clayey matter the results would no doubt be still higher.

The value of B X in the first column of the Tables is the draught of water of the box when floating; it was varied by placing different weights in the box. The value of K C in the second column is the difference between the head of water on the sand and the head of water corresponding to the weight of the box; it is evident that the power of the external forces to overcome the resistance of the sand round the bottom of the box will mainly depend upon this difference. As the water-level in the trough varied with the displacement of the box, this quantity K C was adjusted by adding or subtracting water. The quantities B X and K C were carefully measured by gauges fastened to the four

——	Value of B X, *Fig. 1.*	Value of K C, *Fig. 2.*	Time in
	Inches.	Inches.	Minutes.
Blackheath sand . .	1·11	4·00	8
" " . .	2·11	3·00	18
" " . .	3·11	2·00	32
" " . .	4·11	1·00	50
Woolwich " . .	1·11	4·00	7½
" " . .	2·11	3·00	16
" " . .	3·11	2·00	26
" " . .	4·11	1·00	38

corners of the box. The time in the third column is that elapsed between removing the weight by which the box was forced down into the sand and the floating up of the box.

From these Tables it will be seen that the time during which the box remained sunk is appreciable in all cases, and increased rapidly as K C diminished. The four cases of each Table represent the box under circumstances varying from very light to heavily laden, and the application is as follows:—Take, for example, the last two cases of the first Table, and suppose that when the box had been forced down into the sand with the condition that K C = 1 inch, the open top of the box was only 1 inch above the surface of the water. When K C was 1 inch the box remained down fifty minutes, and when K C was 2 inches the box remained down thirty-two minutes. So that, if after sinking the box, the level of the water were to rise 1 inch in some time between fifty and thirty-two minutes, the water would run into the box and the box would be swamped. This may be considered as corresponding to the case of a ship whose freeboard is equal to about one-fourth of the draught, and if its freeboard were less the case would be worse for the ship.

Let it be assumed that a heavily laden ship is left by a falling tide on a sand-bank, which, from the composition of the sand, or the agitation of its surface by the dash of the waves, or from both causes combined, is more or less a quicksand. As the water falls the ship would press heavily upon the sand, and would sink some depth into it; and the dash of the water for several hours would tend to pile the sand about the bottom of the ship, so that, from these causes, it might be expected that the bottom of the ship would be well bedded in the sand. When the tide rose again and had reached the flotation level of the ship, the ship would not float, but would remain sunk for a considerable time as the tide continued to rise. And, under unfavourable circumstances, it might be held down till swamped, either by the water rising to the level of the deck or by the heavy seas; and even if not directly swamped, it might be stove in or destroyed by the seas, which would act with great effect when the ship was held fast and the tide had risen to near the level of the deck. When once swamped it would of course settle down deeper and deeper into the sand.

The Author is unwilling to push too far conclusions deduced from experiments on a small scale and on a flat-bottomed box. But he is inclined to think that, so far as shape is concerned, a ship would be at a disadvantage compared with the box. For the rounded bottom and keel of a ship would give great facilities for the ship to sink deeply into the sand, and the movement of the water would cause the sand to seal up the ship very closely. At all events, it has been shown by the experiments that there exists in fine sand, with a slight admixture of clayey matter, a capacity for becoming "quick," and holding vessels to a degree which, in combination with other causes, may act powerfully in bringing about some of the terrible and unexplained shipwrecks that so frequently occur on sand-banks.

If in any case it should be thought advisable to make provision for freeing ships from sand-banks which they might be liable to run upon, the plan to be pursued is tolerably clear. All that is wanted is to assist the external water in breaking through the seal of the sand round the bottom, and in re-establishing the full hydraulic pressure. This could be done by forcing water with pumps through holes in the ship's bottom, and would probably soon have the desired effect. On the Thames, when the barges are sometimes swamped by the rising tide after being left on the mud-banks, the practice is to work with an oar under the bottom of the barge when the tide has reached it, and so to let the water get to the bottom. In such cases pumps might probably be introduced with great advantage.

The Paper is illustrated by the two diagrams from which *Figs. 1* and *2* have been engraved.

(*Paper No. 2412.*)

"The Calliope Graving-Dock, Auckland, N.Z."

By JOHN HENRY SWAINSON, Assoc. M. Inst. C.E.

SINCE 1879, the port of Auckland has had the use of a small dock, situated on the southern shore of the harbour, near the wharves and business portion of the town. This dock, until about the time of the wreck of the s.s. "Triumph," at Tiri-tiri, had proved, with few exceptions, though limited to draughts of 13 feet or so, equal to the ordinary requirements of the mercantile shipping visiting the port. On several occasions, however, business has been lost to the city, for want of a larger graving-dock.

It was therefore decided to construct a new dock at Calliope Point, a small promontory midway between Depot Point and the entrance to Shoal Bay, on the northern shore of the waters of the Waitemata, which form the Harbour of Auckland.

With the exception of a certain amount of mud at the entrance, the whole of the excavation was in soft grey sandstone rock, with occasional hard bars and laminated veins.

Vertical fissures were met with here and there, but never of any size to cause trouble; and the whole amount of leakage during the excavation of the rudder-well, the bottom of which was 55 feet below high-water mark and within 140 feet of the tide, was only 83,121 gallons per diem.

On the 5th of November, 1885, being one week less than a year from the signing of the contract, the first shovelful of concrete was laid by the Engineer, Mr. Errington, and the dock was opened in 1887. The dock is 500 feet long, 40 feet wide on the floor, and has a width of 110 feet at the level of the cope, the centre line running S.E. ½ E.

The excavation for this, the greater portion, is 116 feet wide to the level of the rock, or in the rock to a depth of 12 feet from cope level; a bench 5 feet in width reduces it to 106 feet for the next 8 feet in depth; again further reduced by another 5 feet bench, between 20 and 34 feet levels to a breadth of 96 feet; the bottom portion below this again is 76 feet wide, the floor of the excavation being formed concave to a radius of 149 feet, with a fall of 15 inches in the length of the dock, the centre being 47 feet 1 inch below the surface of the ground at the head, and 48 feet 4 inches at the sill.

Down the centre of this runs the 9-inch earthenware drain, laid in concrete in a trench; above this, and all over the floor of the excavation, is a 12-inch layer of ordinary concrete without rubble. This is made in the proportion of 6 to 1 as follows:—2 parts of bluestone broken to 2½-inch gauge, 2 of gravel up to the size of an egg, and 2 of coarse sand to 1 of cement, tested after seven days to stand a strain of 450 lbs. per square inch of cross section. Above this bottom layer of concrete there is an invert built of seven rings of brickwork set on edge in 2 to 1 cement mortar.

This is 60 feet in breadth, and the portion forming the skewback between the edges of the invert and the sides of the excavation is filled in with rubble concrete. The floor of the dock above this invert is formed of rubble concrete, and is convex in shape on the surface, having a camber of 3 inches, all surface water being by this means kept to the sides, where it is conducted by channels to the rudder well and pump wells at the outer end of the dock; the workmen are thereby enabled to repair the keel or bilge of a vessel without standing in water meanwhile.

The total thickness of this floor in the centre is 7 feet, and it has the same fall as the excavation, 3 inches in 100 feet, the centre being 40 feet 1 inch at the head and 41 feet 4 inches at the sill below cope level. Keel blocks, made of pthutukawa, a native timber, heavy, tough, and impervious to the teredo, are placed 5 feet apart, centre to centre, down the dock.

The rudder-well is 21 feet up the centre line from the sill, and is directly opposite the culvert leading to the main pumps. It is 10 feet in diameter, and 15 feet deep, with a hemispherical bottom, and is built of brickwork 2 feet 3 inches in thickness, backed by 12 inches of concrete. Besides the 9-inch pipe discharging the leakage into this well, a 6-inch pipe conveys the surface water from the channel on the S.W. side to it, and a 12-inch earthenware pipe conveys the drainage, &c., thence below the pumping culvert in the direction of the drainage pump. The coping round this well is made of local stone, 12 inches in depth, and is fitted with an iron grating in six sections, supported by one removable iron girder.

At a distance of 20 feet from the centre line on either side rises the first of the seven altars, each 2 feet in height, which form the lower series. The first two of these are 3 feet 7½ inches in width, including the batter, 1 in 8, which is used on all faces, except the caisson ledge and chamber buttresses, throughout the work. The next four are 2 feet 9 inches wide, the upper one or broad altar being 3 feet 9 inches wide. The breadth across the dock at this point, the foot of the deep altar, is 84 feet. Three flights of steps —one central and one 50 feet from either end—give access from the broad altar to the floor on each side of the dock. Above these rises the deep altar, 11 feet 4 inches high, and four 4-feet altars; these, with the exception of the top one, which forms the cope, 3 feet wide, have a width of 2 feet $4\frac{3}{4}$ inches exclusive of the batter. Four flights of steps, one at each corner of the dock, provide a means of communication between the surface and the broad altar.

The head of the dock is formed by a battering wall, 40 feet in height, running at right angles across the dock, broken in the centre by a recess for the bows of vessels which admits of two more flights of steps, from surface to floor, divided by a timber slide down the centre.

All these altars and walls were built of rubble concrete, in layers about 2 feet or more in thickness, and not less than 50 feet in length, composed of rough irregular blocks of basaltic rock, bedded in and surrounded by the ordinary 6 to 1 concrete. The faces of the altars were worked up smooth against frames formed of 2-inch boarding, braced, shored, and weighted on the outside, and accurately set to level line and batter.

The walls at the head and the deep altars required more permanent framing; the ordinary frames were used for the first 2 feet, above which and affixed thereto were uprights, 6 feet by 3 feet, battering inwards, to which a face of 2-inch boarding was attached, the whole being shored up and kept in position, as the wall was built inside, by the gantry.

The centre portion, for a breadth of 41 feet, is struck to a curve of 140 feet radius; this is connected by a bilge curve of 20 feet radius, on either side, to the battering portion 21 feet in height, which gives a total breadth at the surface-level of 80 feet. High-water of ordinary spring-tides gives a depth of 33 feet above the centre of the sill. The rise and fall of the tide averages about 10 feet. The sill is constructed of large blocks of Melbourne stone, laid in eighteen courses, in 1 to 1 Portland cement mortar, well grouted in; this is backed by rubble concrete to the bottom and sides of the excavation, a bulkhead of neat concrete, without any rubble, being inserted behind the caisson quoins to assist in forming a watertight joint; the brick invert of the floor being also returned up to the under-side of the stones composing the inner edge of the sill. The caisson quoins are alternately 5 feet and 4 feet in length, and for the first 24 feet, on either side of the centre line, the courses are from 3 feet 9 inches to 4 feet in depth. Above this the courses are alternately 3 feet and 2 feet 6 inches deep, the coping being a 2-foot course, 3 feet broad on the surface. The first five of the 5-feet caisson quoins project into, and are checked down to the level of the floor of the caisson groove, thus bonding the work together.

The caisson-ledge on the end of the quoins is 16 inches in breadth, and is fine-axed and rubbed down to a perfectly vertical plane to form the joint, with the wood facing of the caisson; beyond this the end of the quoins where visible is checked back 1 inch to ensure the bearing face being clear. With the exception of the ledge, all exposed portions of this stone-work are tooth-axed.

The caisson groove and chamber are 15 feet 6 inches wide upon the floor, which is level, 1 foot 6 inches below the centre of the sill, 3 feet 6 inches in thickness, and 124 feet 6 inches in length; the walls of the chamber batter upwards to within 2 feet of the cope, and are 4 feet 6 inches thick at the level of the floor, where the excavation is in the rock, but are thickened out another 5 feet 6 inches above the level of the rock, the back of the wall being vertical. The end-wall is rather thicker above the rock, in order to furnish a broad-enough foundation for the discharge culvert, which is built in it, and the caisson hauling-gear above. In addition to the vertical walls at the entrance, which are faced with dressed stone from below low-water level, there are two vertical buttresses on each side, and one in each of the inner corners of the chamber, measuring 5 feet on the face. In the centre of the face of each of these side-buttresses, there are grooves into which are dropped totara fenders, projecting 2 inches beyond the face, in order to protect the timber facing of the caisson from being injured by the concrete walls. These buttresses also support horizontal kauri stringers and blocks, to which are attached, over and between the buttresses, cast-iron plates, with rollers, carrying the chain cables of the caisson hauling-gear.

The caisson hauling-gear is fixed on a concrete floor at the north-east end of the caisson chambers, 5 feet below the cope level, and consists of a 6½-inch horizontal shaft, resting on three plummer-blocks, firmly bolted to stones bedded in the floor, with two chain-pulleys, one at each end, and two toothed wheels, worked by worm-wheels on the vertical shafts of capstans, fixed on the timber floor above on the surface level. Two corresponding pulleys, fixed on the vertical walls at the entrance to the chamber, complete the gear.

The western end of the caisson groove forms a portion of the entrance walls, which are 83 feet apart at cope level, and extend seawards to the outer edge of the apron, which is level across, 1 foot higher than the float of the caisson groove, the outer (seaward) edge being bevelled off. Beyond the apron, 31 feet from the sill, these walls have curved returns, at a radius of 10 feet on cope level, and form the eastern and western wing-walls, the former being 98 feet in length in the straight, and the latter 75 feet, including a flight of watermen's steps, which forms a finish in this direction.

The western side of the dockyard is protected from the wash of the sea by a pitched slope. The filling culvert is situated on the western side of the entrance, and is 5 feet 6 inches in height and 4 feet broad, with an invert floor and segmental arched roof.

The mouth of this culvert, which is formed in the masonry of the west wing-wall, is slightly bell-shaped, to make up the amount of waterway lost by the iron grating fixed in a rebate at the outer edge to prevent the influx of timber, rubbish, &c.

Directly opposite the rudder-well, on the east side of the dock, is the mouth of the culvert leading to the pumps; this is covered with an iron grating to prevent chips and other rubbish finding their way to the pumps. The penstock shaft is built of 14-inch brickwork backed with concrete. This portion of the culvert is level, the bottom being the same level as the floor of the main pumps, 51 feet 6 inches below the cope. Just beyond the culvert it opens with a bell-mouth into the suction chamber, 15 feet long by 5 feet wide and 31 feet high; this divides with two openings leading to the two main pump-wells, the lower openings being fitted with a framework of flap-valves.

The lower portion of the pump-wells is 10 feet 6 inches in diameter by 11 feet in height, and contains three large standards, which carry the cage and glands in which the plunger works; a cast-iron floor with valves, &c., divides this from the middle portion, which is 12 feet in diameter, and, including the domed portion, 16 feet in height; this contains the cage, and a wrought-iron bulkhead divides this inlet from the outlet portion. The upper part of these wells is 25 feet deep, with a diameter of 8 feet. Two branch culverts lead from these shafts to the main discharge culvert, which issues through the east wing-wall, at a distance of 17 feet from the end.

Each of the pumps is worked by a separate engine, which, with the pumps themselves, are made from the designs and at the works of Messrs. James Watt and Co., Birmingham. The pumps are calculated to empty the dock from high water, a quantity of over 40,000 tons, in 4½ hours.

There is also a drainage-pump, connected by a 2-foot earthenware pipe laid in concrete with a sump in the main suction-chamber, directly opposite the mouth of the pumping culvert.

The 12-inch pipe from the rudder-well discharges into this. This well is divided into two portions, the lower of which, containing the suction-pipe, is 4 feet 6 inches by 3 feet, and 19 feet 6 inches in height; the invert floor of this is 54 feet below the floor of the engine-house; access to it is obtained by a man-hole in the invert of the upper portion, which is 7 feet by about 6 feet in breadth. This contains the valve-box and pump-barrels, three in number, and 11 inches in diameter, worked by toothed gear from a small vertical engine. The discharge from this well is conveyed by a 15-inch earthenware pipe round the end of the caisson chamber, and issues through the arch of the main discharge culvert, between the penstock and the back of the wing-wall.

This pump is capable of forcing water to a proposed tank on the top of the cliff, 100 feet above the dockyard level, for fire-prevention purposes. Fresh water was obtained from a bore-hole sunk to a depth of 300 feet in the rock on the east side of the dock; this water was used for the boilers during the work, and will be of great service for the permanent boilers of the pumping-engines.

The caisson was designed and made by Messrs. Head, Wrightson, and Co., of Stockton-on-Tees. It is built of wrought-iron, and is 83 feet long on the deck, 15 feet wide over the timber facings, and 41 feet 6 inches deep over all; it is of the box-caisson type, the sides being vertical, 13 feet 6 inches apart; the ends batter downwards at 1 in 8 for 28 feet 6 inches, below which the stem and stern are cut off at an angle of 45°, leaving a length of 48 feet on the floor. The keelpieces are of greenheart 12 feet by 8 feet, one on each side of the bottom of the vessel, and are bolted to 5 inches by 5 inches by ½ inch angle-iron riveted to the floor-plates, which are ⅞ inch in thickness. The caisson is divided into four parts by bulkheads, and has a watertight deck running the full length of the vessel, 16 feet below the covering plates, and a tank floor at the depth of the top plate, for a length of 24 feet in the centre.

There is a 12-inch valve opening from the watertight deck to the sea on each side of the caisson, and also 9-inch ones from the water-ballast tank. A hand-pump on a small deck below the watertight deck connects with either the bilge, the sea on either side, or the watertight deck, and delivers either outside or into the tank, provision being made for affixing hydrants in case of fire.

The Author was appointed by the Harbour Board to superintend the construction of the work, under the direction of the engineer, Mr. William Errington, M. Inst. C. E., and was assisted in the work by Mr. G. Harden.

The Paper is accompanied by two tracings, which may be inspected at the Institution.

(*Paper No. 2454.*)

"Artesian Wells in South Lincolnshire."

By John Charles Gill, Assoc. M. Inst. C.E.

The Peterborough Corporation, having found that the water-supply would soon be inadequate for the rapidly increasing population of that city and borough, decided, upon the advice of the Author, to sink some tube-wells at Wilsthorpe, in Lincolnshire, 14 miles from Peterborough. Three of these artesian wells have now been completed, and they have been so successful in attaining the desired object, at a comparatively small outlay, that a short description of the work may be of interest.

The supply previously available was pumped from a well 5 feet 6 inches in diameter, lined with cast-iron cylinders, and the object of the present work was to augment the quantity that might be drawn from this well by constructing artesian tube-wells discharging into it. A collecting chamber was therefore built, into which the three wells discharge by means of a 6-inch pipe from each, and from this chamber a 12-inch pipe conveys the water to the pump-well.

The wells penetrate the oolite formations, and strike the water-bearing strata at a depth of 53 feet 6 inches from the surface. From this depth the water rises with considerable force, in one instance a stone weighing 6¾ lbs. having been ejected from the top of the tube. Before reaching the water a bed of argillaceous limestone, 8 feet thick, abounding in marine fossils was passed through, and between this rock and the water-bearing strata the estuarine clays, 27 feet in thickness, were pierced. The bore is 7 inches in diameter down to the "water-rock," and is continued a distance varying from 9 to 15 feet in the different wells, with a diameter of 5 inches. A solid-drawn copper tube, 6 inches in internal diameter, and ¼ inch thick, lines the bore down to the water-rock into which it is sealed. To make an absolutely watertight joint between the bottom of the tube and the rock into which it is fixed was a matter of some difficulty, but by repeated trials was in each case effected. The method adopted was to carefully ream the hole in the rock, and then to fit the tube with a conical plug, composed of rings of sheet lead, cut to sizes to fit the reamed hole. The plug was forced down into the hole prepared for it, and after withdrawing two or three times, and carefully examining it to see what alterations were required, either to the plug or to the hole, a good and watertight seal was secured. At the top of each tube a stop-valve is fixed, and by this means the quantity of water drawn from the wells may be regulated according to the consumption in the town. When only one of these extra wells is required, the other two may be shut off entirely, thus preventing the water running to waste.

The daily yield of the wells, gauged about one month after the completion of the last, was as follows:—

	Gallons.
No. 1 well	674,818
No. 2 „	681,108
No. 3 „	810,320
Total	2,166,246

The last boring was completed in September 1888, and the gaugings given were taken on the 12th of November following, the other two borings having then been running some months. From that time down to the present the yield has been fully maintained. During the year 1889, the volume of water pumped from the well into which the borings discharge, was 523,316,410 gallons. At the commencement of the year, the height of the water in the well was 51 feet 6 inches above Ordnance datum, and at the end of the year 52 feet 3½ inches; the rainfall at the pumping-station in 1889 was 22·61 inches. The maximum height of the water-level was 54 feet 10½ inches, on the 8th of June, and the minimum 51 feet 5 inches, on the 26th of October. When the borings were first completed, the water rose to 50 feet 7 inches, but this was after a long-continued term of deficient rainfall; the total for the year 1888 was only 19·57 inches. For some time after completion the water-level continued to fall in the well to 50 feet 1 inch on the 17th of November, 1888, and it has never since been so low. The average height for the year 1889, taken from weekly observations, was 52 feet 10 inches above Ordnance datum. The water in borings is struck at 2 feet 6 inches below Ordnance datum. Ever since the borings were completed they have continued to discharge, each through a horizontal 6-inch pipe, into the collecting chamber; the velocity of discharge, which is always high, varying with the subterranean pressure. This pressure is shown by the height to which the water rises, and as the water-level has never been so low as in the autumn of 1888, the gaugings given may be regarded as the minimum dry-weather yield.

The suction-valves of the pumps are 49·12 feet above Ordnance datum, and, taking the mean of 1889, the water in the well stood 3·71 feet above the suction-valves when the pumps were at rest; but when the stop-valves are regulated so as to give a constant and regular supply, exactly equal to the demand of the pumps, and the pumps are drawing their normal quantity of 86,000 gallons per hour, the water-level in the well is lowered to 4 feet below the suction-valves, and at that level remains constant.

The rainfall registered at the pumping-station is only of comparative value in connection with the yield of the borings, as the outcrop of the oolite rocks from which the water is drawn is many miles away from the wells; but a distinct relation may be traced between the two. Thus the wettest month in last year was May, when 5·01 inches of rain fell; and the maximum height of water in the well was reached on the 8th of June.

The water is of excellent quality for potable purposes, though hard; the hardness, however, is temporary down to 6°. The following is an analysis by Professor Wanklyn:—

The water was of first class purity (organically).
It gave:—

	Parts per million.
Free ammonia	0·14
Albuminiod ammonia	0·05
Total organic matter	2·40

Poisonous metals absent.
One gallon contains:—

	Grains.
Silica	0·5
Carbonate of lime	16·5
Sulphate of lime	2·0
Sulphate of magnesia	4·3
Chloride of sodium	2·9
	26·2

Hardness 23 degrees.
Nitric acid 0·1 grain per gallon.

The total cost of these three wells, including all labour and materials, was £649 13*s.* 11*d.* The Author gives this instance of artesian-well sinking as the most successful that has come under his notice, and believes that the district referred to contains a practically inexhaustible supply of excellent water, which may be obtained at a very moderate cost.

APPENDIX.

The following section gives the thickness and general character of the strata passed through in boring No. 3 well.

	Ft.	Ins.
Surface soil (stiff yellow)	4	7
Clay (dark)	0	6
Peat	1	0
Bed of marine shells	1	0
Limestone (grey)	7	6
Clay (green)	5	8
Hard shale	1	11
Clay (green and dark brown)	27	9
Rock (soft, water-bearing)	3	3
„ (hard, no water)	4	0
„ (soft, water perceptibly increased)	2	0
„ (hard, no water)	0	10
„ (soft, yield of water vastly increased)	3	4
Boring stopped in hard rock.		
	63·4	

(*Paper No. 2418.*)

"On the Probable Errors of Surveying by Vertical Angles."

By Wilfrid Airy, B.A., M. Inst. C.E.

(1.) The method of surveying, dealt with in the present Paper, is that form of tacheometry which consists in determining the distances and levels of points by pairs of vertical angles to known heights on a staff held vertical (Appendix I). The advantages of this method are: (1) That the survey can be effected by means of a simple theodolite, without addition of stadia hairs in the telescope or any other alteration; (2) That there is no question of reading divisions on the staff; if two or three bold well-defined lines be painted on the face of the staff at known heights, and these be further supplemented by lateral projections on the sides of the staff at the same points, there is no difficulty in directing the telescope, and no doubt as to the reading; (3) That from the ease and simplicity of the reading, very long sights can be taken with a small telescope with great accuracy. The field work by this method is at least as rapid as that by any other form of tacheometry, as there are only two angles to be read as against one angle and two staff readings. The reduction work is rather heavier than for instruments furnished with stadia hairs, as in these latter some portion is performed in the instrument itself. The work can be effectually done by a 5-inch theodolite, whereas the tacheometers made for similar work are much larger and heavier. The degree of accuracy of the method forms the subject of the present Paper.

(2.) In order to try the method fairly, the Author set out twenty-four stations in Greenwich Park, forming a circuit of rather more than 1½ mile, and determined the distances and levels of the stations, by the method of pairs of vertical angles, four times over on different days, under different circumstances of weather, and with different adjustments of the instrument. The instrument used was an ordinary 5-inch theodolite. The staff was an ordinary level staff fitted with a small circular level to guide the staff-holder in keeping it vertical. The ground was very favourable for trying the method, as some parts were steep and others level; the circuit involved a rise and descent each of 118 feet. The distance of the successive stations apart varied from about 4 to 6 chains; the average distance was 341 feet. The theodolite was set up at the alternate stations so that the reading distances also varied from 4 to 6 chains. At these distances there was no difficulty in accurately sighting the large red figures on the staff. The staff was 16 feet long, and the usual plan was to read at 2 feet and at the top of the staff, thus intercepting a height of 14 feet, but when obstacles intervened any other readings were adopted, but always of whole feet.

(3.) In order to ascertain the errors for distance between the stations, the line was carefully chained and levelled in the usual way. The results of the chain and level measurements were taken as approximately correct, and the errors were found by subtracting the theodolite results from the chain and level results. These errors were sometimes + and sometimes —, and the average error of distance between the stations without regard to sign was 0·925 foot. The average distance between the stations was 341 feet, and there were, therefore, 15·48 such distances to the mile. Consequently the probable error of distance in a mile is $0{\cdot}925\sqrt{15{\cdot}48} = 3{\cdot}64$ feet. In like manner the average error, without regard to sign, of the difference of level between the even stations (the theodolite being set up at the odd stations) was found to be 0·12 foot, and as there were 7·74 such double distances to the mile, the probable level error in a mile is $0{\cdot}12 \times \sqrt{7{\cdot}74} = 0{\cdot}33$ foot.

(4.) The above-mentioned errors for distance are chiefly due to the combination of small errors, including errors of verticality of the staff, of directing the telescope, of levelling the theodolite, and of reading the vertical circle. Each of these errors directly affects the measurement of the vertical angles, and the sum of them may, therefore, be regarded as an angle error. Now the average distance being 341 feet, and the average distance error being 0·925 foot it will be found from the expression for the distance error in Appendix I. that the average angle error is about 23″, or rather less than half a minute of arc. There appears to be no reason why the same angle error should not apply to all cases where the staff can be clearly viewed.

(5.) Referring again to the expressions given in Appendix I. for the distance error and level error, due to a small angle error, it will be seen that the distance error varies as the square of the distance, and on level ground the level error varies as the simple power of the distance; on steep ground the level error will vary more nearly as the square of the distance. If, therefore, the average distance between the stations had been taken at 170½ feet instead of 341 feet, the average distance error would have been $\frac{0 \cdot 925}{4} = 0 \cdot 23$ foot. There would have been twice as many such distances to the mile as before, so that the probable distance error in a mile would have been $0 \cdot 23 \times \sqrt{2 \times 15 \cdot 48} = 1 \cdot 28$ foot. The level error (assuming the ground to be level) would have been $\frac{0 \cdot 12}{2} = 0 \cdot 06$ foot between the even stations, and there would have been $2 \times 7 \cdot 74 = 15 \cdot 48$ double distances to the mile; so that the probable level error in a mile would have been $0 \cdot 06 \times \sqrt{15 \cdot 48} = 0 \cdot 24$ foot. In this way it is seen that to reduce the errors it is only necessary to shorten the distances, but of course this is gained at the cost of reduced speed of work, and the most advantageous distance in any case must be determined by the circumstances.

(6.) The closing errors of a circuit (Appendix III.), though valuable as a general check on the work, cannot be relied on to give the probable distance errors per mile for two reasons: first, because they are affected by the small errors of the azimuth angles; secondly, because, if in consequence of errors of adjustment of the instrument or other causes the distances were systematically too long or too short, the errors would cancel one another at different parts of the circuit, and the closing error might be very small, although the total length of the line might be considerably in error. As a matter of fact, the average terminal error of the four series of observations was only 2·84 feet in the circuit of 1½ mile. Therefore, the probable errors to be relied on are those given in paragraph (3).

(7.) The probable error of distance in 1 mile being 3·64 feet, and that of level being 0·33 foot (paragraph 3), the probable errors in 25 miles would be 18·20 feet and 1·65 foot respectively. And these errors would be the same whether the ground were rough or smooth, provided that the distances between the successive stations did not greatly differ from an average of 5 chains. The distance error in a chained survey would probably greatly exceed 18 feet in 25 miles if the ground were rough, because, in rough ground the chain lengths have a systematic tendency to be short. The level error in 25 miles obtained by ordinary levelling ought to be much less than 1·65 foot. Therefore, in running long lines in a rough country, the distances would be most accurately determined by the theodolite method, but it would be advisable to correct the levels of the main traverse line by ordinary levelling, as the work proceeds; the same corrections could then be applied to the levels of the points determined right and left of the line. The levelling would also afford a safeguard against accidental errors.

(8.) The chief advantage of this and of other methods of tacheometry is the rapid and approximate determination of the positions and levels of a number of points right and left of a main traverse line. The degree of accuracy that may be expected for single readings at different distances may be estimated thus: The average error of distance (the distance being about 5 chains) has been shown to correspond to an angle error of rather less than half a minute of arc (paragraph 4). And the same angle error may be taken for any distance at which the staff could be clearly sighted. If then the ground be level, the intercepted height on the staff 14 feet, and the angle error half a minute of arc, the average distance error at 5 chains would be 1·13 foot, at 10 chains 4·5 feet, and at 20 chains 18·1 feet. The corresponding level error at 5 chains would be 0·048 foot, at 10 chains 0·096 foot, and at 20 chains 0·192 foot. With regard to the effect of steep ground on the errors of distance and level, see Appendix I.

(9.) In Appendix I. are given the formulas necessary for the computation of the distances and levels, with instructions for the application of them. In Appendix II. are given specimens of the Field Book and Reduction Book in the manner found most convenient. In Appendix III. is shown the method of determining the closing errors of a circuit.

The Paper is illustrated by diagrams from which the *Figs.* in the text has been prepared.

[APPENDIXES.

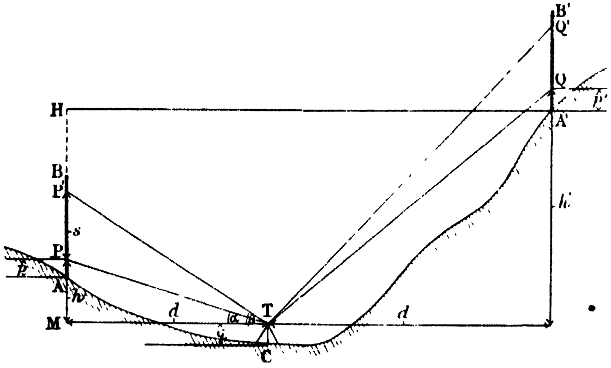
APPENDIXES.

APPENDIX I.

INVESTIGATION of FORMULAS, and RULES for APPLICATION.

In the diagram below (*Fig. 1*) let A A^1 be two stations, and T the position of the telescope of a theodolite at a height T C $= q$ above the ground. Let A B be a graduated staff held truly vertical on the ground at the point A. Produce the vertical line B A to meet the horizontal through T in M. Let P P^1 be two well-defined points on the staff to which the telescope is directed in succession, such that A P $= p$, and P P^1 $= s$, both known quantities. Let (α) be the angle subtended at T by P M, and (β) the angle subtended by P^1 M, these being measured angles. Let T M $= d$ and P M $= h$. Then from the triangles P T M, P^1 T M, we have P^1 M $= d \tan \beta$, and P M $= d \tan \alpha$. Therefore P P^1 $= d (\tan \beta - \tan \alpha)$ or $d = \frac{s}{\tan \beta - \tan \alpha}$; also P M $= d \tan \alpha$, or $h = \frac{s \tan \alpha}{\tan \beta - \tan \alpha}$. With a table of natural tangents it is easy to compute the values of (d) and (h). In the above formulas $\tan \beta - \tan \alpha$ will always be positive; (h) will be positive or negative according as (α) is positive or negative, *i.e.* according as the reading for (α) is above or below the horizontal plane through the telescope.

Fig. 1.



In the same way (d^1) and (h^1) can be determined, and from the figure the difference of level of the points A A^1 = A H = H M − A M $= h^1 - p^1 - (h - p) = h^1 - h - (p^1 - p)$. In this formula (h) and (h^1) must have their proper signs. If (p^1) be taken equal to (p), as would generally be the case, the difference of level of the points A A^1 will be $(h^1 - h)$.

From the figure it is seen that the horizontal distance of C from A $= d$, and the vertical distance of C from A $= h - p + q$. This last will be only approximate, as (q) can only be measured approximately.

If (m) be the reduced level of the point A, the reduced level of the point $A^1 = m + \{h^1 - h - (p^1 - p)\}$, and that of the point $C = m - (h - p + q)$. In both these formulas (h) and (h^1) must have their proper signs. The reduced level of the point C will be only approximate.

From the expressions for (d) and (h) it may be shown that a small angle error will affect the horizontal distance by nearly the same amount, whether the ground be steep or level, but will affect the vertical height very much more on steep ground than on level ground. This may be seen as follows:— $d = \frac{s}{\tan\beta - \tan\alpha}$, and $h = \frac{s\tan\alpha}{\tan\beta - \tan\alpha}$, and suppose that the aggregate errors of observation are equivalent to an angle error of ($\delta\alpha$), so that (α) becomes ($\alpha + \delta\alpha$). Let (δd) and (δh) be the errors of (d) and (h) due to the angle error ($\delta\alpha$). Then it can be easily shown that $\delta d = \delta\alpha \cdot \frac{d^2}{s} \cdot \frac{1}{\cos^2\alpha}$, and $\delta h = \delta\alpha \cdot \frac{d^2}{s} \cdot \frac{1}{\cos^2\alpha}\left(\frac{s}{d} + \tan\alpha\right)$. The average distance between the stations in the survey was 341 feet, and the average error of distance corresponded to an angle error of about 23″, or rather less than half a minute of arc.

Let $d = 340$ feet, $s = 14$ feet, and $\delta\alpha = \frac{1}{2}'$ of arc; then the following are the values of (δd) and (δh) for different values of (α):—

(α) in Degrees.	δd Per Single Distance (d) in Feet.	δh Per single Distance (d) in Feet.	δh Per Double Distance ($d + d^1$) in Feet.
0	1·20	0·05	0·07
5	1·21	0·16	0·22
10	1·24	0·27	0·38
15	1·29	0·40	0·56
20	1·36	0·55	0·78

The figures of the last column are formed from those of the third column by multiplying them by $\sqrt{2} = 1·414$. They are the probable errors of the difference of level of the fore and back stations.

From the expressions for the errors of (d) and (h) is seen the importance of keeping (d) tolerably small and (s) as large as possible, especially in steep ground.

APPENDIX II.

COPY of PAGE of FIELD BOOK.

No. of Station where Theodolite is placed.	Object viewed.	Azimuth Angle of Object.	Compass Reading.	Intercepted Portion of Staff, in Feet. (s)	Vertical Angle of Lower Reading on Staff. (α)	Vertical Angle of Upper Reading on Staff. (β)	Height of Telescope above Ground, in Feet. (q)
	Station.	° ′	°		° ′	° ′	
11	10	..	261	2–16	89 45	92 10½	4 90
11	12	180 24	81	2–16	89 3½	91 55½	
12	11	..	261½				
12	13	178 41	80				
13	12	..	260	2–16	89 56	92 31	4 82
13	14	83 5	343½	4–16	89 12	90 52	
14	13	..	163				
14	15	180 9	343				
15	14	..	163¼	2–16	90 35	92 32	4 88
15	16	181 56	346¼	2–16	83 25½	85 44	
16	15	..	165				
16	17	176 44	342				
17	16	..	161½	2–16	95 3	97 21	4 88
17	18	166 11	328	2–16	86 23	88 25	
18	17	..	148½				
18	19	116 57	266				

Notes.

(1.) The compass reading forms a check on the azimuth angles.

(2.) The figures under (s) are the lower and upper points of the staff reading.

(3.) As regards the values of (α) and (β); before commencing work the instrument is adjusted so that when the telescope is horizontal the reading on the vertical circle is 90° 0′.

(4.) There is, of course, a page for remarks.

Copy of Page of Reduction Book.

No. of Station where Theodolite is placed.	Object viewed.	Azimuth Angle of Object.	Horizontal Distance of Object from Telescope, in Feet. $(d) = \frac{s}{\tan\beta - \tan\alpha}$	Height of Lower Reading on Staff above Telescope, in Feet. $(h) = \frac{s \tan\alpha}{\tan\beta - \tan\alpha}$	Difference of Level of Object Station and Back Station. $h^1 - h - (p^1 - p)$.	Difference of Level of Theodolite Station and Back Station. $-(h - p + q)$.	Reduced Level of Object Station. $m + \{h^1 - h - (p^1 - p)\}$ and $m - (h - p + q)$.
	Station.	° ′					
11	10	..	330·7	−1·44	..	..	215·63
11	12	180 24	279·7	−4·60	−3·16	..	212·47
12	11	..	..	..	..	..	..
12	13	178 41	..	..	..	−2·46	210·01
13	12	..	310·3	−0·36	.. −5·40 −2·00	..	212·47
13	14	83 5	412·5	−5·76	−7·40	..	205·07
14	13						
14	15	180 9	..	..	..	−7·06	198·01
15	14	..	411·0	+4·18	..	..	205·07
15	16	181 56	344·3	−39·68	−43·86	..	161·21
16	15	..					
16	17	176 44	..	..	..	−33·33	127·88
17	16	..	344·6	+30·45	..	..	161·21
17	18	166 11	393·6	−24·88	−55·33	..	105·88
18	17	..					
18	19	116·57	..	..	..	−4·27	101·61

Notes.

(1.) The first three columns are filled in from the Field Book.
(2.) There must also be a page for remarks.

APPENDIX III.

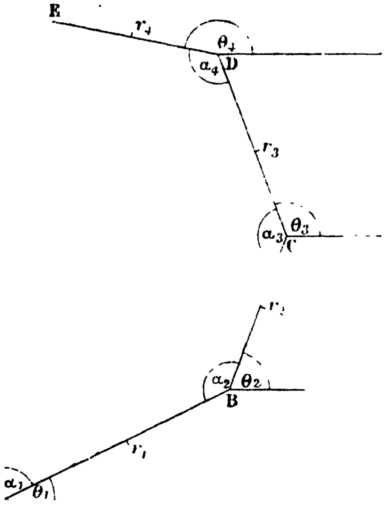
Method of determining the closing Errors of a Circuit.

It is assumed that the interior angles of the polygon are measured as carefully as possible. They are then to be checked by the usual rule, viz. that the sum of all the interior angles of any closed rectilineal figure, together with four right-angles, are equal to twice as many right-angles as the figure has sides. If, as ought always to be the case, the angle error is very small, the closing error of the circuit can be determined in the following manner:—Let the polygon (*Fig. 2*) be started at A, and let A B be the first side; let A X, A Y be rectangular axes of co-ordinates chosen arbitrarily so that the angle Y A B = α_1; let α_2 α_3 α_4, &c., be the successive internal angles of the polygon, and θ_1 θ_2 θ_3 θ_4, &c., be the angles which the successive sides make with lines parallel to A X. Then the co-ordinates of B are $r_1 \sin \theta_1$ and $r_1 \cos \theta_1$, and the co-ordinates of C are $r_1 \sin \theta_1 + r_2 \sin \theta_2$ and $r_1 \cos \theta_1 + r_2 \cos \theta_2$, &c., &c. So that for a figure of (n) sides the co-ordinates of the last point are—

Co-ordinate parallel to A Y $= r_1 \sin \theta_1 + r_2 \sin \theta_2 +$, &c., $+ r_n \sin \theta_n$.
Co-ordinate parallel to A X $= r_1 \cos \theta_1 + r_2 \cos \theta_2 +$, &c., $+ r_n \cos \theta_n$.

In the case of a closed figure these quantities represent the closing errors in the directions parallel to A Y and A X respectively.

Fig. 2.



The angles θ_1 θ_2 θ_3, &c., are determined in terms of α_1 α_2 α_3, &c., in the following manner:—

$$\theta_1 = 90^\circ - \alpha_1$$

$$\theta_2 + \alpha_2 + 180^\circ - \theta_1 = 360^\circ \therefore \theta_2 = 3 \times 90^\circ - \overline{\alpha_1 = \alpha_2}$$

$$\theta_3 + \alpha_3 + 180^\circ - \theta_2 = 360^\circ \therefore \theta_3 = 5 \times 90^\circ - \alpha_1 \overline{+ \alpha_2 + \alpha_3}.$$

&c. &c.

and generally

$$\theta_n = (2\,n - 1) \times 90^\circ - \alpha_1 + \alpha_2 + \alpha_3 + \quad . \quad . \quad . \quad + \alpha_n.$$

If, as might sometimes be convenient, the co-ordinates are measured in the direction of A B, and at right angles to A B, it is only necessary to make $\alpha_1 = 0$ in the above formulas.

(*Paper No. 2447.*)

"Wire-Ropes."

By Andrew Stevenson Biggart, Assoc. M. Inst. C.E.

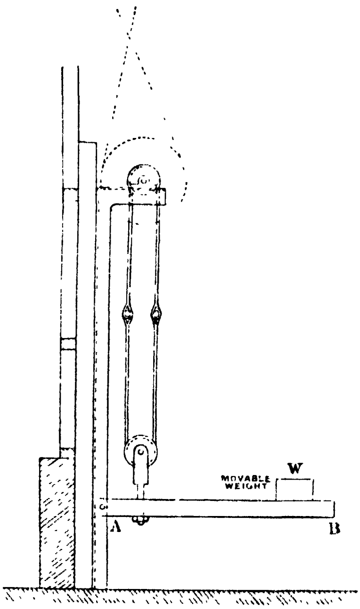
The experiments described in this Paper were undertaken with the object of discovering some practical means of selecting the best form of wire-rope to be employed in the erection of the Forth Bridge. When it is stated that upwards of 60 miles of wire-ropes were employed in connection with that undertaking, the importance of the investigation becomes obvious. At an early stage in these observations it became evident that by extending the tests a little much useful information might be gained, as to the behaviour of wire-ropes under varying conditions, and results of these experiments are now laid before the Institution.

Wire-ropes are now put to various uses, and the work demanded from them differs to a wide extent. The form of the rope, therefore, must be suited to the purpose for which it is required. The principal causes which lead to the destruction of wire-ropes are: the wearing away of the outer surface of the outside wires, the rubbing of the wires against one another, and fatigue of the steel brought about when the rope is worked over a pulley relatively too small. A good example of ropes which give out by the wearing away of the outside wires is found in tramway cables, while deterioration of the last kind is seen in ropes working on crane barrels or in pulley blocks. In the case of tramway cables no practical consideration calls for small pulleys to guide the rope round street corners or similar leads. It is true that the carrying pulleys must be comparatively small; but these affect the life of the cable very little. Recognizing that the wear of the wires, due to the friction of the car-grippers, is in most instances very serious, while the stress due to bending the ropes round the pulleys is comparatively small, it is at once apparent that in this case a stiff rope may be used, having wires of a moderately large diameter, so as to allow of a sensible reduction in size, before the outer wires are worn completely away. On the other hand it will be observed that the conditions under which ropes work over small pulley blocks are the reverse of those in the case of tramway cables. Small pulleys are, for convenience, a practical necessity, and in consequence the wires must be of a small diameter, so that the stress due to bending round the pulleys may be kept within moderate limits. Flexible ropes are in these instances a matter of absolute necessity, and the tabulated experiments in this Paper refer almost wholly to this class of wire ropes.

TESTING MACHINE.

The form of apparatus adopted in the tests was of the simplest kind, and was designed to wear out the ropes in a manner similar to that which takes place in practice. On the testing machine were two pulleys, over which the wire-rope was stretched. The lower pulley was attached to a hinged lever A B, *Fig. 1.* This lever was loaded with a movable weight W to give the requisite tension. As the rope band was not long enough to allow of satisfactory splicing, and thus continuous running over the pulleys, it was made in two equal sections, joined together by eyes and shackles; a reciprocating motion was given to the rope thus strained, by an automatic arrangement (not shown) connected with

Fig. 1.



the upper pulley. A record was kept of the number of bends sustained by each rope put under test, thus affording means for comparison.

GENERAL CHARACTERISTICS.

In Table I, part 1, are detailed some of the general characteristics of different makes of wire-rope. In all cases the ropes were of flexible make, with the exception of those 3 inches and 4 inches in circumference. This stiffer type of rope was never called on to perform any other work than the carrying of dead loads, or acting as wind ties. It was ultimately discarded for the flexible ropes, which were as cheap, when allowance was made for the extra strength of the crucible steel and increased handiness.

A cross-section of the ropes presents a series of six circles clustered around a central one. Each of the six outer circles is in contact with the inner one and those on each side. The inner circle, or core, is generally wholly made of hemp, although in some instances several wires are placed in the heart of it, for the primary purpose of preventing the rope from twisting. The outer circles represent the strands, which are wound round the core, from left to right, at a slight angle. The strands, in some cases, have a hemp core, in others a wire one; but of whatever substance the core is, the wires of the strand are laid over it, sometimes from right to left, at other times from left to right.

It will be observed that the ideas of rope manufacturers as to the proper tensile strength of wire vary very much, ranging from 68 tons to 103 tons per square inch, in the same class of flexible rope.

A point of considerable importance is the difference between the aggregate strength of a number of steel wires, when tested singly, and that of the same wires when tested in the form of a rope. The loss of strength in the rope (although this cannot be stated with certainty, as the manufacture is in some cases more perfect than in others) seems to be about 10 per cent. The ropes tested were in short lengths, and as they were bent round a solid eye, and spliced at each end, it is probable that in ropes of the length usually employed this difference would be considerably reduced.

If the breaking-strength in direct tension of these steel-wire ropes is compared with that of hemp-ropes and chains, it will be found that to obtain equal strength, the weight of the hemp-rope must be about three times, and that of the chain five times the weight of the wire-rope.

The second part of Table I deals with the general features of ropes obtained for testing purposes. The results of running these ropes, and the individual wires of these ropes, over pulleys of various diameters, and the modulus, and limit of elasticity of the wires, are given in succeeding Tables. Special attention may, however, here be directed to Nos. 4 and 7 of Table I, part 2. No. 4 is a sample of Bessemer ingot wire of 45 tons, and No. 7 of steel wire of 163 tons ultimate tenacity per square inch.

WIRE.

The wire used in these flexible ropes was understood to be made from crucible steel. The tests adopted for determining the quality of the wire were those of tension and torsion.

The average tensional strength of the wire used, ascertained by direct pull on the individual wires, was from 80 tons to 90 tons per square inch.

The torsion, or ductile test, is the number of complete revolutions, or twists, which wire of a given length will stand before fracture. A piece of good wire, of 20 B. W. G. and 8 inches in length, should stand from seventy to eighty such turns. The mode of ascertaining this value is to place a piece of wire in an apparatus capable of gripping and rotating one end, while the other is held fixed to a point 8 inches distant, so that this length of 8 inches is the only free part of the wire where twisting can take place.

The variation in tensional strength of contiguous pieces of steel wire, both galvanized and ungalvanized, is very little (see Table II). In the case of galvanized the variation is only 3 per cent., while in the ungalvanized wire it is about 8 per cent.; in both cases not more than what may be looked for in any ordinary piece of steel.

The results of the tests of ductility on Table III are more striking. Contiguous pieces of six different series, representing three makes of ropes were tested. In the first series of six wires, all galvanized, the average was 68 twists, while the lowest was 31, and the highest 96. In series 2 of this Table it will be observed, that in one case the number of twists withstood by pieces taken within a few inches of each other, from the same wire, varies from 23 to 58. The average of this series is 62, the lowest being 23, and the highest 90. The last set of six wires, in this Table, shows an average of 92 turns, while the lowest gives 75, and the highest 103. In this case the wires were ungalvanized, which accounts for the greater ductility of the steel. Another point brought out in this same Table, as also in Table II, is, that the results are practically uniform, whether the wire is, prior to twisting, put under a slight initial tension or left quite free. No appreciable difference is observed in the average result, whether the wires are twisted slowly, or so quickly as to become heated. Some interesting results of experiments with galvanized and ungalvanized wires, both annealed and unannealed, are brought out in Table II. In the case of galvanized wire, the loss in ductility, when annealed, is very marked. The average, when annealed, is 57·5 twists, against 85·5 when unannealed. The gain in ductility, in the case of ungalvanized wire when annealed, is, on the other hand, as striking. Thus the annealed ungalvanized wire shows an average of 178·5 twists, as against 101·5 when unannealed. The loss in tensional strength, due to annealing, in galvanized and ungalvanized wire, is practically the same; the galvanized falling from 207 lbs. to 118 lbs., and the ungalvanized from 217 lbs. to 120 lbs. in wire of 20 B. W. G. Experiments made to determine the modulus, and limit of elasticity, and the extension under direct pull, are recorded in Table IV. The results were ascertained on lengths of 100 inches. The modulus of elasticity is shown to vary from 22,400,000, in the case of the Bessemer iron wire, to 35,000,000 in piano steel wire. The ordinary crucible steel wire, shows a modulus of 33,000,000, the permanent set commencing at about half the ultimate tenacity. Permanent set took place in the case of Bessemer iron wire at about 12 tons per square inch, or about one quarter of the ultimate tenacity. No guide is furnished as to the point beyond which a permanent set takes place, by ascertaining the ultimate breaking-strength of a wire. Permanent set may take place at from 25 per cent. to 80 per cent., and even more, of the breaking strength. In galvanized and ungalvanized wire, the ratio of extension up to the elastic limit is practically the same. The extension at the breaking point, of good crucible steel wire, may be taken at about 3 per cent. in lengths of 100 inches.

PULLEY TESTS.

Reference has already been made to the series of experiments in which ropes were run over pulleys until destruction ensued.

The pulleys ranged from 5¼ to 24 inches in diameter, and flexible steel wire-ropes of 1¾ inch circumference were employed.

The results of the tests are given in Table V.

The failure of the ropes was in all cases occasioned by the individual wires gradually giving way, one by one. In no case were the outside wires of the ropes severely worn away, although in a few instances, where the pulleys were large and the number of bends endured before breakage consequently greater, wearing away took place to a considerable extent. Before attempting to describe these experiments in detail, it will be well to examine, briefly, the conditions prevailing in the rope, while passing over the pulleys.

To bend a $1\frac{3}{4}$-inch rope, composed of seventy-two wires, 20 B.W.G., over a small pulley, where the wires are free to move, is an easy matter. It will, nevertheless, take more than seventy-two times the power required to bend one of the wires to the same curve. If the wires composing the rope are soldered together at any point near to where the bend takes place, the power required to make the bend will be sensibly increased. The resistance may be thus augmented till it approaches that of a solid bar, as when the wires are all securely joined to one another along their entire length. The movement of the strands, as a rope is bent, is easily observed by the naked eye. If a rope be conceived straight, and under a longitudinal stress, it is apparent that the strands and wires will be held to one another by friction, due to the twisting around each other received during manufacture. The value of this friction may be illustrated by considering the strength of a splice. A splice of 40 feet in length will easily resist the full strength of an 80-ton tension steel wire, when in the form of a rope. A splice may thus give a resisting power of 2 tons per lineal foot.

If a rope, under tension, is bent round a pulley, the resistance will be that due to the bending of the individual wires, as beams, plus that due to the increased stiffness given to the wires, as a whole, on account of their being laid into one another.

The resistance to bending, on the part of the individual wires, is highest where small pulleys are employed, and bending in these cases is often the direct cause of by far the greatest separate longitudinal stress in the wires themselves.

If a wire is bent to a radius R, the bending moment at any point is, $M = \frac{E\,I}{R}$, where I is the moment of inertia of the section of the wire, and E is the modulus of elasticity. The stress consequently due to bending is $f = \frac{E\,I}{2\,R}$.

In Table V the stress due to bending a 20-B.W.G. wire, with E equal to 32,000,000, round pulleys of various sizes is given. It will be observed that the stress, due to bending alone, in a wire 20 B. W. G. diameter, when bent round a pulley 10½ inches diameter, is 50 tons. This at once shows that the usual ropemakers' allowance, in determining the size of the pulley as six circumferences of the rope, irrespective of the diameter of the wire, is beyond the limits of prudence, and if acted on will assuredly give a short life to a hard-wrought rope. It is evident that the diameter of the pulley should not be made to depend simply upon the circumference of the rope, but that the size of the component wires should also be taken into consideration.

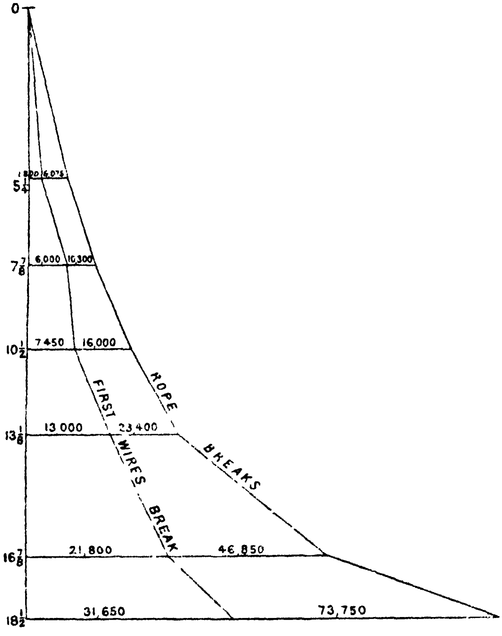
Where small pulleys are employed, the principal stress in a rope, and the one to be kept most in view, is not that due to the uniform direct tension caused by the working load, but is the bending stress just referred to. Other stresses than those named are present in a rope, such as, the stress due to centrifugal force, that due to the bending of the wires while being laid on the strands, and also when the strands are being formed into the rope. These last stresses may be discarded, as they happen only once in the life of a rope. If, however, they were recurrent each time the rope passed over the pulley, they would be destructive ones, as they amount to that which produces a permanent set in the individual wires.

As will be observed from Table V, the full series of tests really begin with the 10½-inch pulley, although on each of the two smallest pulleys one test was made. In the case of these small pulleys the working stress in the wires was so far beyond the elastic limit that no appreciable length of life was obtained. The number of bends reached with the 10½-inch and 16⅞-inch pulleys respectively, shows no marked difference, although, in the case of the first, the working stress was beyond, while in that of the second it was practically within the limit of elasticity. The diagram given in *Fig.* 2 shows, graphically, the increase in the life of the rope as the diameter of the pulleys is increased. No idea is furnished as to the point where the working stresses change from beyond to within the elastic limit, owing to the regularity of the increase. This may be partly accounted for by the fact that the wires, when running over comparatively small pulleys, get much cut up where they cross one another, owing to the longitudinal movement in the rope already referred to. This cutting action is easily observed in the wires, by examining a rope that has run for some time. The effects can be gauged, to a certain extent, by the torsion test; thus, a 1¾-inch rope was run over a 10½-inch pulley 16,000 times, with a load equal to $\frac{1}{10}$ of its breaking weight, when it was found the torsion test gave the following average results:—

Ductility of wires before running commenced	64
„ „ near fracture	6
„ „ at centre of wires not broken	3
„ „ midway between fracture and splice . .	30

No doubt much of the deterioration, in the rope referred to, was due to fatigue of the steel, but a large share must also be ascribed to the cutting action of the wires against one another. It is worthy of note that, after having been run over the pulleys, the value of the part of the rope nearest the middle, where that cutting movement was greatest, was as 3 is to 30, compared with another part, where the bending was the same but the movement of the wires against each other less. This cutting action is also demonstrated by comparing the number of sustained bends, of a given rope, with the number borne in the case of individual wires, of a similar quality of steel. This same experiment may be

Fig. 2.



compared with experiment No. 1, on Table VIII, where 15,000 bends were given to wires of the same quality over a pulley of the same diameter. The result was to reduce the value of the steel from 77 to 32. The difference between 3, in the test above referred to, in which the rope stood 16,000 bends, and 32 in this one with the wire, is the loss due to the cutting action of the wires.

This cutting action may be modified to a very great extent by a judicious system of oiling. The oil should be capable of permeating the rope.

As is to be expected, the advantages due to oiling are more clearly marked when the ropes are worked within the elastic limit of the steel. For example (see Table V), Smith's ordinary rope, gave, on the 10½-inch pulley, when unoiled, 16,000 bends before breaking, but when oiled, 38,700. This same make of rope when running over the 24-inch pulley, stood, when unoiled, 74,000 bends against 386,000 when oiled. In this instance the movement of the wires would be less than in the case of the tests with the 10½-inch pulley. In these tests the wires composing the strands were laid from left to right, and the strands themselves, from right to left. The wires, as a consequence, cross one another rather severely.

In Table V are also the results of some experiments with a type of rope known as "Lang's lay," in which both the wires and the strands are laid from left to right. The result is that the wires composing a strand lie into the wires of the adjoining strands, against which they bear; thus, in a great measure, securing a longitudinal bearing surface throughout. A better bearing surface is also given the outside of the rope, and the cutting action is reduced to a minimum. This rope gave, on the 10½-inch pulley, when unoiled, 53,000 revolutions, against 107,600 when oiled, and 38,700 for the first type of rope, when oiled also.

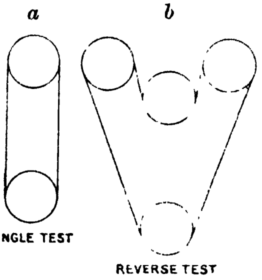
In connection with the bending of ropes over pulleys, experiments were also conducted to ascertain the difference between running over single pulleys, where the bending was all in one direction, (as in *Fig. 3, a*) and over pulleys placed as in *Fig. 3, b*, so that the rope, while running, was subjected to reverse stresses. The results are given in Table VI, and are instructive, as showing that generally the life of a rope, when bent in one direction only, is twice as long as when working under reverse stresses.

Other interesting points are brought out. For example, the longer life of No. 1 rope over No. 0, although the quality of the steel was the same. In the latter the wires cross one another abruptly, while in the former the bearing is almost longitudinal.

The experiments in Table VII were undertaken for the purpose of comparing the life of single wires with that of ropes composed of similar wires. The general result is clearly this, that the life of the wire, when running alone over a pulley, is much greater than when in a rope, and running over the same pulley. Perhaps the most striking feature, in the results of the experiments recorded in this Table, is the failure of the wires in several cases, as the working stress was well within the elastic limit. In all cases failure was due to the fatigue of the steel, no appreciable wear being observable on the surface of the wires.

Test No. 4 is also remarkable. The wire was of a low quality of Bessemer iron, and worked beyond the elastic limit, yet the result attained was better than from any of the other wires, with one exception. It even excelled high class plough steel, which in this Table bore a poor record.

Figs. 3.



The experiments on Table No. VIII reveal the effects of fatigue, or severe work, on a 20 B.W.G. wire running over a $10\frac{1}{2}$-inch pulley. In every case the result is similar. The effects are: loss of tensional strength, loss of extension, and decrease in the value of the torsion test. In this Table, again, the Bessemer iron wire shows to advantage.

From the information gained by the foregoing experiments, some conclusions may be deduced as to the best type of rope to use under varying circumstances. Thus a $1\frac{3}{4}$-inch flexible rope to run over, say, ordinary pulley blocks, should be made of ungalvanized steel wire, of about 20 B.W.G., with a tensional strength of about 80 tons per square inch. Hemp cores, oiled during manufacture, should be placed in the strands, as well as in the centre of the rope. The wires and strands ought to be laid in the same direction. Further, when in use, the rope should be regularly oiled, to nullify as much as possible the cutting action of the wires, and at the same time prevent rusting. In a case such as that named, the diameter of the pulleys, will, owing to practical considerations, always be small, so that what is wanted is the best compromise. The user will be content to pay for a greater number of ropes, rather than have the pulleys made of such a diameter as theory and practice points out to be necessary for a long life; and in doing so he will be the gainer, as in almost all cases increased economy is obtained through the use of comparatively small pulleys.

Theoretical considerations would point to a 6-foot 6-inch pulley as a minimum, for a 1¾-inch rope made of 20 B.W.G. wire. Allowance is here made for the effect of reverse stresses, which are always present in the case under consideration. While a general conclusion may be arrived at, such as that given, for a wire rope working over pulley blocks under certain conditions, it must be clearly seen that before determining what is the best form of rope for any given work, the conditions of economy under which it is to be used require to be clearly understood and provided for.

The Paper is accompanied by the diagrams, from which the *Figs.* in the text have been prepared.

[APPENDIXES.

APPENDIXES.

TABLE I.—PART 1.

Maker.	Circumference.	Quality of Steel.	Weight per Fathom.	Strands.						Breaking - Weight of Rope, average of two.	Breaking - Weight of Wires, average of six.	Breaking - Weight of Wires per Square Inch.	No. of Wires multiplied by their Average Breaking-Weight.	Loss per cent.	Ductile Test.	Remarks.
				Number of Strands.	Number of Wires in one Strand.	Total Number of Wires.	Diameter and Area of Wires.									
							Diameter.	B. W. G.	Area.							
	Ins.		Lbs.				Inch.		Inch.	Tons.	Lbs.	Tons.	Tons.			
T. & W. Smith	1¾	Crucible	2·05	6	12	72	0·036	20	0·0010	6·80	212	94·64	6·82	0·3	73	Ordinary flexible.
”	1¾	”	2·55	6	24	144	0·036	20	0·0010	10·47	190	84·82	12·22	14·3	78	Special ”
”	1¾	”	2·10	6	12	72	0·036	20	0·0010	6·62	230	102·68	7·39	10·3	92	Ungalvanized.
Bullivant & Co.	1¾	”	2·00	6	12	72	0·040	19	0·0013	5·85	200	68·68	6·43	9·0	66	Oiled in manufacture.
Dixon, Corbett & Newall	1¾	”	..	6	..	..	0·040	19	0·0013	9·85	228	78·29	..	..	46	Lang's patent lay.
R. S. Newall	1¾	”	..	6	19	114	0·036	20	0·0010	5·75	210	93·75	..	..	83	Ungalvanized.
T. & W. Smith	2¼	”	4·55	6	24	144	0·040	19	0·0013	14·85	263	90·30	16·91	12·2	60	Ordinary flexible.
”	2¾	”	6·80	6	24	144	0·048	18	0·0018	21·90	367	91·10	23·59	7·2	..	” ”
”	3	Bessemer	..	6	7	42	0·110	11½	0·0095	14·10	1,123	52·77	21·05	33·0	..	
”	4	”	..	6	7	42	0·144	9	0·0163	26·10	1,693	46·36	31·74	17·8	..	

TABLE I.—PART 2.—DESCRIPTION OF SPECIAL 1¾-INCH ROPES SENT BY MESSRS. T. & W. SMITH FOR TESTING PURPOSES.

No.	Description.	Strands.	Total Number of Wires.	Wires.			Breaking-Weight of Wire, average of Six; Tons per Square Inch.	Torsion Test with 30 lbs. Initial Tension.		
				Diameter.	Wire Gauge.	Area.		Minimum.	Maximum.	Average of 36.
				Inch.		Square Inch.				
1	Lay similar to Lang's; crucible steel . .	6	72	0·036	20	0·00100	100	27	86	71
2	Patent improved steel wire	6	72	0·036	20	0·00100	83	85	226	108
3	Plough steel wire . .	6	72	0·036	20	0·00100	120	80	104	95
4	Iron wire	6	72	0·040	19	0·00130	45	4	210	69
5	Crucible steel . . .	6	54	0·049	18	0·00188	86	43	62	54
6	" " . . .	6	108	0·028	22	0·00060	92	73	120	99
7	Piano wire	..	..	0·028	22	0·00060	163	16	107	69

TABLE II.—TESTS OF TWO SINGLE WIRES, ONE GALVANIZED AND ONE NOT GALVANIZED, BOTH BEFORE AND AFTER BEING ANNEALED.

The wires were supplied by Messrs. T. & W. Smith, and were of crucible steel, No. 20 B. W. G., similar to those used by them in their 1¾-inch rope. In the torsion test the wires were fixed in the machine for twisting them with and without initial tension alternately.

No.	Not Annealed.						No.	Annealed.					
	Galvanized.			Ungalvanized.				Galvanized.			Ungalvanized.		
	30 lbs. Tension.	No Tension.	Breaking-Weight.[1]	30 lbs. Tension.	No Tension.	Breaking-Weight.[2]		30 lbs. Tension.	No Tension.	Breaking-Weight.	30 lbs. Tension.	No Tension.	Breaking-Weight.
			Lbs.			Lbs.				Lbs.			Lbs.
1	82	79	205	101	101	207	1	66	57	118	161	163	123
2	93	86	209	95	96	225	2	63	72	121	157	153	115
3	82	81	208	99	110	219	3	52	84	..	176	184	118
4	86	91	205	102	95	220	4	61	33	119	178	192	121
5	76	89	207	104	102	225	5	77	57	118	183	180	122
6	93	85	209	98	100	220	6	36	42	118	129	190	120
7	89	87	210	108	105	219	7	73	72	118	177	202	119
8	84	85	204	106	100	214	8	44	29	116	180	189	125
9	85	76	210	101	105	214	9	..	..	..	197	188	121
10	84	94	204	103	103	212	10	..	..	..	174	188	
11	83	85	209	97	108	210	11	..	..	..	203		
12	83	91	208	96	95	220	12	..	..	..	174		
Average	85	86	207	101	102	217	Average	59	56	118	174	183	120
	Average 88·5			Average 101·5				Average 57·5			Average 178·5		

[1] Difference between highest and lowest 3 per cent.

[2] Difference between highest and lowest 8 per cent.

TABLE III.—VARIATION IN DUCTILITY OF ADJACENT LENGTHS OF SIX WIRES, TAKEN FROM DIFFERENT ROPES 1¾-INCH IN CIRCUMFERENCE.

The length of the part of the wire subjected to twisting was in all cases 8 inches, and the wires were tested with and without a tension of 30 lbs. per wire alternately.

Description.		First Wire.		Second Wire.		Third Wire.		Fourth Wire.		Fifth Wire.		Sixth Wire.	
		30 lbs. Tension.	No Tension.	30 lbs. Tension.	No Tension.	30 lbs. Tension.	No Tension.	30 lbs. Tension.	No Tension.	30 lbs. Tension.	No Tension.	30 lbs. Tension.	No Tension.
(1.) Wires taken from Messrs. T. & W. Smith's ordinary flexible 1¾-inch rope. Galvanized wire, 20 B. W. G. . . .	First length	50	..	60	..	86	..	62	..	84	..	86	
	Second "	..	35	..	66	..	86	..	59	..	76	..	76
	Third "	31	..	55	..	78	..	72	..	80	..	77	
	Fourth "	..	50	..	66	..	96	..	65	..	70	..	83
	Fifth "	37	..	53	..	70	..	70	..	80	..	80	

Average with tension = 67. Average without tension = 69.

Description.		First Wire.		Second Wire.		Third Wire.		Fourth Wire.		Fifth Wire.		Sixth Wire.	
		30 lbs. Tension.	No Tension.	30 lbs. Tension.	No Tension.	30 lbs. Tension.	No Tension.	30 lbs. Tension.	No Tension.	30 lbs. Tension.	No Tension.	30 lbs. Tension.	No Tension.
(2.) Wires taken from Messrs. Bullivant & Co.'s flexible 1¾-inch rope. Galvanized wire, 20 B. W. G.. .	First length	39	..	70	..	64	..	77	..	70	..	90	
	Second "	..	42	..	70	..	63	..	66	..	62	..	87
	Third "	38	..	42	..	57	..	50	..	64	..	80	
	Fourth "	..	23	..	63	..	59	..	64	..	61	..	83
	Fifth "	57	..	61	..	43	..	58	..	63	..	89	
	Sixth "	..	58	..	60	..	63	..	45	..	59	..	90

Average with tension = 62. Average without tension = 62.

Description.		First Wire.		Second Wire.		Third Wire.		Fourth Wire.		Fifth Wire.		Sixth Wire.	
		30 lbs. Tension.	No Tension.	30 lbs. Tension.	No Tension.	30 lbs. Tension.	No Tension.	30 lbs. Tension.	No Tension.	30 lbs. Tension.	No Tension.	30 lbs. Tension.	No Tension.
(3.) Wires taken from Messrs. T. & W. Smith's ungalvanized 1¾-inch rope. Galvanized wire, 20 B. W. G. . . .	First length	92	..	88	..	87	..	94	..	94	..	93	
	Second "	..	91	..	103	..	95	..	75	..	90	..	102
	Third "	89	..	97	..	91	..	88	..	91	..	96	
	Fourth "	..	87	..	98	..	87	..	97	..	91	..	95
	Fifth "	90	..	98	..	95	..	94	..	91	..	96	
	Sixth "	..	94	..	93	..	93	..	95	..	98	..	83

Average with tension = 92. Average without tension = 92.

TABLE IV.

The limit and modulus of elasticity of—

1st. Galvanized crucible steel wire 20 B. W. G.

2nd. Iron wire 19 B. W. G.

Both supplied by Messrs. T. & W. Smith as being similar to those used in their ordinary 1¾-inch ropes; also of—

3rd. Piano wire No. 22 B. W. G. = 0·028-inch diameter.

Each the average of two tests.

Length of wire experimented on = 100 inches.

Tension.	Crucible Steel.		Iron Wire (Bessemer).		Piano Wire (Ungalvanized).	
	Extension.	Permanent Set.	Extension.	Permanent Set.	Extension.	Permanent Set.
Lbs.	Inch.	Inch.	Inch.	Inch.	Inch.	Inch.
10	0·03	Modulus of elasticity 33,000,000 lbs.	0·025	Modulus of elasticity 22,400,000 lbs.	0·025	Modulus of elasticity 35,000,000 lbs.
20	0·06		0·060		0·075	
30	0·09		0·100		0·125	
				Permanent set commences at 12 tons per square inch.		
40	0·12		0·130	0·01	0·150	
50	0·15		0·175	0·02	0·200	
60	0·18		0·220	0·035	0·275	
70	0·21		0·260	0·05	0·325	
80	0·24		0·310	0·075	0·375	
90	0·28		0·360	0·10	0·425	
100	0·32		0·420	0·125	0·500	
						Permanent set commences at 74 tons per square inch.
110	0·36	..	0·500	0·15	0·525	0·005
		Permanent set commences at 60 tons per square inch.				
120	0·39	0·05	0·600	0·20	0·580	0·01
130	0·43	0·01	0·800	0·36	0·640	0·02
140	0·46	0·02	134 lbs. =46 tons	..	0·720	0·03
150	0·51	0·03	..	..	0·770	0·04
160	0·55	0·04	..	..	0·850	0·07
170	0·59	0·06	..	..	0·900	0·08
180	0·64	0·08	..	..	1·000	0·12
190	0·74	0·30	..	..	1·120	0·16
200	1·40	0·42	..	..	1·200	0·20
210	2·50	1·75	..	..	1·350	0·30
228= 102 tons	5·50	..	..	..	220=1·60 225=1·75 =163 tons	0·42 broke

TABLE V.—THE LIFE OF FLEXIBLE STEEL WIRE-ROPES WHEN RUNNING OVER IRON PULLEYS.

The figures in the Table give the number of times the rope passed over the various pulleys before breaking.

All ropes 1¾ inch in circumference. Load on rope, 14 cwt.

Maker.	Diameter of Pulleys in Inches.						
	5¼	7⅞	10½	13⅛	16⅞	18¾	24
	Rope breaks.	Rope breaks.	Rope breaks.	Rope breaks.	Rope breaks.	Rope breaks.	Rope breaks.
Messrs. T. & W. Smith's ordinary rope	6,075	10,300	16,000	23,400	46,800	72,700	74,100
Messrs. Bullivant & Co.'s	..	..	34,800	..	..	100,200	336,600
Lang's patent lay .	..	..	53,100	85,200	..	392,500	
Messrs. T. & W. Smith's ordinary rope, oiled . .	..	..	38,700	37,300	..	163,200	386,100
Lang's patent lay, oiled	..	..	107,600	142,700			

CALCULATED STRESS IN TONS PER SQUARE INCH ON WIRES OF 20 B. W. G. WHEN BENT OVER THE PULLEYS.

E = 14,700 tons.

Diameter of Pulley.	Stress due to Bending.	Stress due to direct Tension.	Total Stress.
Inches.			
5¼			
7⅞			
10½	50·0		
13⅛	40·0		
16⅞	31·4		
18¾	28·2		
24	22·0		

In calculating the stress due to bending, no allowance has been made for the error in the ordinary formula when applied to solid sections.

TABLE VI.—LIFE OF ROPES WHEN RUNNING OVER 24-INCH DIAMETER PULLEYS.
Load on rope in all cases, 14 cwt.

Description of Rope.	Total Number of Bends before the Rope breaks when	
	Running over one 24-inch Pulley. (*Fig. 3, a*).	Running over three 24-inch Pulleys. (*Fig. 3, b*).
No. 0. Smith's ordinary crucible steel, wire and strands laid in opposite directions	74,100	51,000
No. 1. Smith's rope, wire and strands laid in the same direction	..	126,000
No. 2. Smith's patent improved steel, wire and strands laid in opposite directions	96,000	57,000
No. 3. Smith's plough steel, wire and strands laid in opposite directions	109,000	54,000
No. 4. Smith's iron wire, 19 B. W. G., wire and strands laid in opposite directions	66,000	32,000
No. 5. Smith's crucible steel, 18 B. W. G., wire and strands laid in opposite directions	87,000	47,400
No. 6. Smith's crucible steel, 22 B. W. G., wire and strands laid in opposite directions	111,000	48,700

With the exception of Nos. 4, 5, and 6, the wires composing all ropes were 20 B. W. G.

TABLE VII.—LIFE OF SINGLE WIRES TAKEN FROM ROPES NO. 2 TO NO. 6, RUNNING OVER PULLEYS 5¼, 10½, AND 24 INCHES DIAMETER, AND LOADED IN EACH CASE WITH ONE-TENTH OF THEIR BREAKING-WEIGHT.

Description.	Number of Bends before Breaking.			Calculated Stresses.[1]		
	Average of two Tests (*Fig.* 3, *a*).			Pulley.	Stress from Bending.	Stress from Direct Tension.
	Pulley 5¼ inches.	Pulley 10½ inches.	Pulley 24 inches.	Diameter inches.	Tons per Sq. Inch.	Tons per Sq. Inch.
No. 2. Improved steel; wires 20 B. W. G.	15,200	104,000	453,000	5¼		
				10½	50	8·3
				24	22	8·3
No. 3. Plough steel; wires, 20 B. W. G.	22,000	128,700	181,300	5¼		
				10½	50	12
				24	22	12
No. 4. Iron wire; wires, 19 B. W. G.	12,800	83,500	355,800	5¼		
				10½	38	4·5
				24	17	4·5
No. 5. Crucible steel; wires, 18 B. W. G.	..	45,000	123,400	5¼		
				10½	70	8·6
				24	30	8·6
No. 6. Crucible steel; wires, 22 B. W. G.	20,000	..	232,000	5¼		
				10½	40	9·2
				24	17	9·2

In the above Table the following values for E have been taken:—for steel wire, 14,700 for all ropes; for iron wire, 10,000.

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[1] In calculating the stress due to bending, no allowance has been made for the error in the ordinary formula when applied to solid section.

Table VIII.—Test of Single Wires after running over 10½ inch Diameter Pulleys, and Loaded in each case with one-tenth of their Breaking-Weight.

Wire, from—	Number of Bends.	Tensional Strength.		Ultimate Elongation per cent.		Torsional Test.	
		Before Running.	After Running.	Before Running.	After Running.	Before Running.	After Running.
		Tons per Sq. Inch.	Tons per Sq. Inch.	Average of 2.	Average of 2.	Average of 36.	Average of 36.
No. 1. Smith's crucible steel, 20 B. W. G. . . .	5,000	100	..	..	..	71	77
	10,000	100	100	5·5	3·4	71	56
	15,000	100	98	5·5	3·0	71	32
No. 2. Improved steel (crucible) 20 B.W.G.	30,000	83	76	1·0	1·2	108	103
	60,000	83	..	..	..	108	69
	90,000	83	..	..	..	108	18
No. 3. Iron wire (Bessemer) 20 B. W. G.	15,000	45	45	0·75	1·75	69	29
	30,000	45	45	0·75	1·25	69	31
	45,000	45	42	0·75	0·75	69	18

(*Paper No. 2471.*)

"Notes on a Short Method of Distributing Triangulation Errors."

By Alfred Edward Wackrill, Assoc. M. Inst. C.E.

The example taken is the Ghati-Maihar double polygon, Fig. No. 50 of the Great Indus Series, N.W. Quadrilateral of the G.T.S.I. The solution of this polygon by the rigorous method used in the Indian and other triangulations, is shown in the official report on the Great Triangulation, and the results obtained by the two methods will be compared.[1]

The only advantage claimed for the short method about to be explained over the rigorous method is its shortness. But it is believed that the results obtained are sufficiently accurate for countries in which a highly scientific triangulation is not obtainable for want of funds, or which, from their limited extent or isolated position, require only an internal harmony for the purposes of survey. The shortness of the method consists not only in the smaller amount of space covered by the calculations, but also in the checks that can be applied to the working at every step.

The problem is this: The 24 internal angles of the polygon have been observed on the spheroidal surface of the earth, and there must be a small amount of unavoidable error in every, or nearly every observation that has been taken. The 24 angles have now so to be adjusted that

(1) The sum of the corrected angles of every triangle in the figure shall equal 180°, after the necessary allowance has been made for the "spheroidal excess" caused by the relative inclination of the vertical axes of the observing instruments at the various stations.

(2) The sum of the angles about each of the central stations, C and F, shall equal 360°, without allowing for spheroidal excess.

(3) The length of any line in the figure shall have the same value, by whatever route it is calculated.

(4) The corrections to the angles shall be the smallest possible for the attainment of the above conditions.

The 4th condition is attained by the "Method of Least Squares," a development of the theory of probabilities. By this method, each observed quantity receives what is known as a "weight" depending on the number of times the quantity has been observed, the mutual agreement between the observations, the degrees of excellence of the instruments used in taking the observations, and the ascertained skill of the various observers. A value which is supposed to be very near the truth has a larger weight assigned to it than one which is less accurate. And since the less accurate measurements require a larger correction than the others, the error to be assigned to each angle in a figure like the one under consideration must be a function of the reciprocal of the weight.

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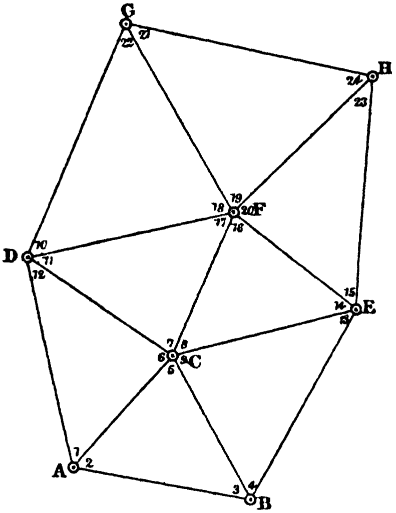
[1] G. T. Survey of India, vol. ii., p. 210. G. T. Survey of India, vol. iii., Great Indus Series, p. 3—д.

Fig.



The order of procedure here adopted is, first to adjust the individual triangles so as to fulfil condition (1), and having regard to the reciprocal weights; secondly, to adjust independently each of the two pentagons which compose the figure, so as to satisfy conditions (2) and (3) without disturbing condition (1); and, lastly, to combine the two pentagons into one double polygon, fulfilling conditions (1), (2), and (3), and satisfying condition (4) as nearly as may be.

Table I shows in the first column the numbers of the angles as indicated in the *Figure*. In the second column are the observed angles grouped according to the eight triangles which make up the polygon. The third column gives the reciprocal weights of the angles, and the fourth the corrections for spherical excess.

The first correction for triangular error is shown in the fifth column, and is obtained by dividing the error of each triangle in proportion to the reciprocal weights of its angles. Thus, if A, B, C, be observed angles of any triangle, S_a, S_b, S_c, their spheroidal excesses, R_a, R_b, R_c their reciprocal weights, and K_a, K_b, K_c their corrections in column five,

$$K_a = 180 - \{A+B+C-(S_a+S_b+S_c)\}\frac{R_a}{R_a+R_b+R_c};$$

$$K_b = 180 - \{A+B+C-(S_a+S_b+S_c)\}\frac{R_b}{R_a+R_b+R_c}.$$

Column 6 gives the algebraic sum of the quantities in 4 and 5; column 7 gives $A - S_a + K_a$, $B - S_b + K_b$, &c. In column 7 the degrees and minutes are omitted for brevity, as the seconds only are affected. The log sines of the angles in column 7 are shown in column 8.

Each of the eight triangles of the polygon has now received the treatment accorded to a triangle which forms by itself a link in a chain of polygons, and triangles. In fact, if the observations taken to and from either one of the stations D or E, were for any reason not obtainable, the values of K_a, K_b, K_c, &c., would have to be accepted as the most probable, and the lengths of the sides calculated without further correction. The fact (which will shortly appear) that there is error still uncorrected in one or more of the angles, proves that the value calculated as the most probable is not always (it may even be hazarded, is never) the true value. This is due to the imperfection of a knowledge of all but comparatively few of the causes which go to produce error.

The sum of the angles at the central stations C and F should by condition (2) be 360°. The following Table shows the deduction of the central errors.

No.	Observed Angles.	Corrections from col. 5, Table I.	No.	Observed Angles.	Corrections from Table I.
	° ′ ″			° ′ ″	
5	70 46 4·42	+0·20	16	75 19 36·34	−0·43
6	82 9 54·90	−0·05	17	55 23 59·48	+0·22
7	78 30 38·05	+0·14	18	71 44 52·50	−0·12
8	52 59 46·09	−0·32	19	75 46 51·68	+0·03
9	75 33 36·44	−0·03	20	81 44 40·17	−0·31
	359 59 59·90 360 0 0·00	−0·06		360 0 0·17 360 0 0·00	−0·61
	Error = −0″·10	−0·10 −0·16		Error = +0·17	+0·17 −0·44

The central error now to be dealt with is:—(the sum of the angles at the central station) − 360° + (the algebraical sum of the triangular corrections in Table I, column 5).

In order that any side may have the same value by whatever route it is calculated, the following equations must be satisfied:

$$\frac{\text{Sin }(2)\ \sin (12)\ \sin (17)\ \sin (14)\ \sin (4)}{\text{Sin }(1)\ \sin (11)\ \sin (16)\ \sin (13)\ \sin (3)} = 1 \quad . \quad . \ (\text{I})$$

$$\frac{\text{Sin }(11)\ \sin (22)\ \sin (24)\ \sin (15)\ \sin (8)}{\text{Sin }(10)\ \sin (21)\ \sin (23)\ \sin (14)\ \sin (7)} = 1^{1} \quad . \quad (\text{II})$$

Taking first the southern pentagon:—The first two groups in Table II show, in the second column, the angles of the left member of equation (I), those of the numerator in the first group, those of the denominator in the second.

The angles are taken from column seven, Table I.

To satisfy equation (I) the sum of the log sines of the first group must equal the sum of the log sines of the second group. It appears from the second column that this is not yet satisfied, as there is a difference of 0·0000002,8.[2]

Now if the amount 2,8 is distributed among the logs of the two groups so as to make the *corrected* ΣL_l = *corrected* ΣL_r (where ΣL_l denotes the sum of the log sines of the left-hand angles, ΣL_r denotes the sum of the log sines of the right-hand angles), equation (I) will be satisfied. If at the same time it can be contrived to make the algebraic sum of the ten angular corrections, corresponding to the logarithmic corrections so introduced, equal to − 0·16 second, the sum of the angles will be diminished in equation (I) by 0·16 second, and the sum of the central angles thereby increased by that amount, so satisfying the central equation.

Since for very small increments to angles the log sines increase in proportion to the angular increments, write in column 4 the difference of log sine for 1 second opposite each angle (taking care in

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[1] The above formula follows from the well-known fact that in any plane triangle whose angles are A, B, C, and lengths of opposite sides a, b, c, respectively

$$\frac{a}{\text{Sin A}} = \frac{b}{\text{Sin B}} = \frac{c}{\text{Sin C}}.$$

[2] In this investigation the zeros at the commencement of small decimal fractions will be omitted, the comma serving to locate the position of the digits. Thus 0·000 0002,8 will be written 2,8. In the same way, to save space and avoid unnecessary repetition, the last five figures of an 8-place logarithm will be frequently used to represent the logarithm.

general to place the negative sign to the difference for any angle over 90°). Now by making the logarithmic increments proportional to the numbers in column 4, equal increments are made in the angles. In order that the increments may be proportional to the reciprocal weights, opposite the angles place in column 5 their reciprocal weights and in column 6 write the products, difference for 1 second × reciprocal weight.

Using the following notation:—

d_1, d_2, &c., = the differences for 1 second in the log sines of the angles (1), (2), &c.,
R_1, R_2, &c., = the reciprocal weights of (1), (2), &c.,
l and r subscript, = the left-hand or first group, and right-hand or second group respectively,
x = the increment about to be made in the sum of the log sines of left-hand angles,
y = same for right-hand angles,
p = corresponding increment to sum of left-hand angles,
q = ,, ,, ,, ,, right-hand angles,

it will be seen that, in general,

$$p = \frac{\Sigma\,\mathrm{R}_l}{\Sigma\,(d \times \mathrm{R})_l}\,x \qquad \text{and } q = \frac{\Sigma\,\mathrm{R}_r}{\Sigma\,(d \times \mathrm{R}_r)}\,y,$$

and $\Sigma\,\mathrm{L}_l + x = \Sigma\,\mathrm{L}_r + y$.

Substituting values for symbols, there are the two equations for x and y[1]—

$$x - y = 2{,}8,$$

$$\frac{0\cdot 73x}{11{,}3} + \frac{0\cdot 50y}{7{,}9} = -\,0\cdot 16,$$

giving $x = 0{,}14 \qquad y = -\,2{,}66,$

and $p = 0''\cdot 01 \qquad q = -\,0''\cdot 17.$

In column 7 are placed the logarithmic corrections—

$$\frac{d_2\,\mathrm{R}_2\,x}{\Sigma\,(d \times \mathrm{R})_l},\ \frac{d_{12}\,\mathrm{R}_{12}\,x}{\Sigma\,(d \times \mathrm{R})},\ \cdot\ \cdot\ \cdot\ \cdot\ \cdot\ \cdot\ \cdot\ \cdot\ \cdot$$

$$\frac{d_1\,\mathrm{R}_1\,y}{\Sigma\,(d \times \mathrm{R})_r},\ \frac{d_{11}\,\mathrm{R}_{11}\,y}{\Sigma\,(d \times \mathrm{R})_r},\ \cdot\ \cdot\ \cdot\ \cdot\ \cdot\ \cdot\ \cdot\ \cdot\ \cdot$$

and in column 8 the logarithms of column 3 corrected by those amounts.

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[1] See note 2 on previous page.

In column 9 are the angles corresponding to the log sines in column 8.

The third and fourth groups show the same process applied to the northern pentagon. The two pentagons have now been independently adjusted internally, so as to nearly satisfy all condition equations, as shown below.

THREE SOUTHERN-PENTAGON TRIANGLES.

	°	′	″		°	′	″		°	′	″
2	57	27	44·07	1	53	43	28·01	13	46	36	48·20
3	51	46	11·72	12	44	6	37·59	4	57	49	35·92
5	70	46	4·21	6	82	9	54·40	9	75	33	35·88
Sum	180	0	0·00		180	0	0·00		180	0	0·00

TWO TRIANGLES COMMON TO BOTH PENTAGONS.

			S.	N.	Mean.				S.	N.	Mean.
	°	′	″	″	″		°	′	″	″	″
7	78	30	37·72	37·55	37·63	8	52	59	45·35	45·34	45·34
11	46	5	23·08	23·13	23·11	16	75	19	35·45	35·52	35·49
17	55	23	59·20	59·32	59·26	14	51	40	39·20	39·14	39·17
	180	0	0·00	0·00	0·00		180	0	0·00	0·00	0·00

THREE NORTHERN-PENTAGON TRIANGLES.

	°	′	″		°	′	″		°	′	″
10	55	56	1·04	21	47	17	26·33	23	42	25	24·89
22	52	19	7·37	24	56	55	42·55	15	55	49	55·75
18	71	44	51·59	19	75	46	51·12	20	81	44	39·36
	180	0	0·00		180	0	0·00		180	0	0·00

CENTRAL ANGLES.

	°	′	″	Spheroidal excess.		°	′	″	Spheroidal excess.
5	70	46	4·21	0·44	16	75	19	35·52	0·44
6	82	9	54·40	0·50	17	55	23	59·32	0·50
7	78	30	37·72	0·50	18	71	44	51·59	0·83
8	52	59	45·35	0·44	19	75	46	51·12	0·77
9	75	33	35·88	0·56	20	81	44	39·36	0·57
	Spher. excess		2·44	2·44		Spher. excess		3·11	3·11
	360	0	0·00			360	0	0·02	

The amount × 0·02 second, the error of the sum of the north central angles, will be dealt with below. Now, since the angles of the two triangles common to both pentagons are equally probable from whichever pentagon they are obtained, their means may be taken as their final values. This will slightly affect both the central equations and both the side equations. The errors thus introduced will have to be dispersed as shown in Table III in a manner precisely similar to that already explained in Table II.

The central angles now stand as follow (seconds only being given).

	South. ″		North.	
	4·21		16	35·49
	54·40		17	59·26
	37·63		18	51·59
	45·34		19	51·12
	35·88		20	39·36
Spherical excess	2·44	Spherical excess		3·11
	59·90			59·93

$\therefore p + q$ for Table III $= -0·10$ $\therefore p + q$ for Table III $= -0·07$

The quantity $-0·07$ includes, of course, the error of $+0·02$ second existing in the north central equation before the combination of the pentagons. This error is no doubt due to the fact that a Table of 7-place logarithms (Shortrede's) was used by the Author, no better one being available.

In Table III a reciprocal weight of zero is assigned to each of the angles common to the two pentagons (namely, 7, 11, 17, 16, 14, and 8).

The following are the final values of the angles, which now satisfy all the equations of condition. The degrees and minutes being unaffected are here omitted for brevity.

	″		″		″		″
2	44·04	1	27·99	13	48·18	7	37·63
3	11·70	12	37·58	4	35·92	11	23·11
5	4·26	6	54·43	9	35·90	17	59·26
	0·00		0·00		0·00		0·00

	″		″		″		″
8	45·34	10	1·03				
16	35·49	22	7·37				
14	39·17	18	51·60	5	4·26	16	35·49
				6	54·43	17	59·26
	0·00		0·00	7	37·63	18	51·60
				8	45·34	19	51·16
21	26·29	23	24·87	9	35·90	20	39·38
24	42·55	15	55·75	Spher. excess	2·44	Spher. excess	3·11
19	51·16	20	39·38				
	0·00		0·00		0·00		0·00

COMPARISON OF RESULTS.

Table IV needs very little explanation. Column 6 gives the errors found by the short method explained above, and the next column the errors found by the rigorous method of the Indian Triangulation.[1]

The mean difference between the two sets of errors is shown to be 0·0226 second.

If a = the base of the polygon (the side A B).
b = the side of continuation (side G H).

Log sin (5) = 9·975 0601,8
„ (13) = 9·861 3761,7
„ (16) = 9·985 5994,2
„ (23) = 9·829 0503,1
„ (21) = 9·866 1712,9
49·517 2573,7

Log sin (2) = 9·925 8467,1
„ (4) = 9·927 5965,9
„ (8) = 9·902 3253,5
„ (15) = 9·917 7133,4
„ (19) = 9·986 4865,8
49·659 9685,7
49·517 2573,7

Log. $\left(\frac{b}{a}\right)$ = 0·142 7112,

(From Indian book) log a = 4·950 8025,6
∴ log b = 5·093 5137,6
∴ b = 124026·29 feet.
In Indian book b = 124026·19 „

Difference 0·10

Excess per mile . . . $= \frac{0·1}{23·49}$ foot.
„ „ . . . = 0·00425 „
„ „ . . . = 0·051 inch.

GENERAL REMARKS.

The foregoing method may be applied to any figure forming a link in a triangulation chain such as those used in the Great Trigonometrical Survey of India. On completing a circuit of such chains, some misclosure is sure to be found, and a series of extremely small corrections is required affecting every link throughout the circuit. This class of correction cannot be dealt with in this place, but may perhaps form the subject of a future Paper.

The Paper is accompanied by a diagram, from which the *Fig.* in the text has been prepared.

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[1] G. T. Survey of India, vol. ii. p. 216.

[APPENDIX.

APPENDIX.—TABLE I. SHOWING THE APPLICATION OF THE FIRST CORRECTION, NAMELY, THAT FOR TRIANGULAR ERROR.

1.	2.	3.	4.	5.	6.	7.	8.
No of Angle.	Observed Angles.	R. W.	Corrections.			Corrected Angles.	∠ Sines.
			Spheroidal Excess.	Triangular.	Total.		
	° ′ ″					″	
2	57 27 43·19	0·27	−0·44	+1·31	+0·87	44·06	9·925 8467,3
3	51 46 11·71	0·10	−0·44	+0·49	+0·05	11·76	9·895 1640,5
5	70 46 4·42	0·04	−0·44	+0·20	−0·24	4·18	
	179 59 59·32		−1·32	+2·00	+0·68	0·00	
1	53 43 28·63	0·15	−0·50	−0·07	−0·57	28·06	9·906 4325,0
6	82 9 54·90	0·11	−0·50	−0·05	−0·55	54·35	
12	44 6 38·15	0·14	−0·50	−0·06	−0·56	37·59	9·842 6365.0
	180 0 1·68		−1·50	−0·18	−1·68	0·00	
4	57 49 36·55	0·13	−0·57	−0·06	−0·63	35·92	9·927 5966,0
9	75 33 36·44	0·06	−0·56	−0·03	−0·59	35·85	
13	46 36 48·84	0·09	−0·57	−0·04	−0·61	48·23	9·861 3762,6
	180 0 1·83		−1·70	−0·13	−1·83	0·00	
7	78 30 38·05	0·09	−0·50	+0·14	−0·36	37·69	9·991 2088,8
11	46 5 23·49	0·08	−0·50	+0·12	−0·38	23·11	9·857 5900,2
17	55 23 59·48	0·15	−0·50	+0·22	−0·28	59·20	9·915 4705.9
	180 0 1·02		−1·50	+0·48	−1·02	0·00	
10	55 56 2·18	0·07	−0·82	−0·22	−1·04	1·14	9·918 2346,0
18	71 44 52·50	0·04	−0·83	−0·12	−0·95	51·55	
22	52 19 8·83	0·23	−0·82	−0·70	−1·52	7·31	9·898 4087,0
	180 0 3·51		−2·47	−1·04	−3·51	0·00	
19	75 46 51·68	0·10	−0·77	+0·03	−0·74	50·94	
21	47 17 27·26	0·14	−0·77	+0·04	−0·73	26·53	9·866 1717,6
24	56 55 43·28	0·08	−0·77	+0·02	−0·75	42·53	9·923 2388,4
	180 0 2·22		−2·31	+0·09	−2·22	0·00	
8	52 59 46·09	0·06	−0·44	−0·32	−0·76	45·33	9·902 3253,3
14	51 40 39·86	0·04	−0·44	−0·22	−0·66	39·20	9·894 6115,4
16	75 19 36·34	0·08	−0·44	−0·43	−0·87	35·47	9·985 5994,1
	180 0 2·29		−1·32	−0·97	−2·29	0·00	
15	55 49 56·50	0·10	−0·56	−0·22	−0·78	55·72	9·917 7133,1
20	81 44 40·17	0·14	−0·57	−0·31	−0·88	39·29	
23	42 25 25·70	0·07	−0·56	−0·15	−0·71	24·99	9·829 0505,8
	180 0 2·37		−1·69	−0·68	−2·37	0·00	

TABLE II.—SHOWING THE APPLICATION OF THE SECOND CORRECTION, VIZ., THAT FOR CENTRAL AND SIDE ERRORS.

Southern Pentagon.

1.	2.	3.	4.	5.	6.	7.	8.	9.
No. of Angle.	Angles from Table I, Col. 7.	Logarithmic Sines.	Difference for 1″ = *d*.	Reciprocal Weight = R.	*d* × R.	Correction to Logarithm.	Corrected Logarithm.	Corrected Angles.
	Left hand.							
	° ′ ″							″
2	57 27 44·06	9·925 8467,3	+13,5	0·27	+3,6	+0,1	8467,4	44·07
12	44 6 37·59	9·842 6365,0	+22,0	0·14	+3,1	0	6365,0	37·59
17	55 23 59·20	9·915 4705,9	+14,5	0·15	+2,2	0	4705,9	59·20
14	51 40 39·20	9·894 6115,4	+17,0	0·04	+0,7	0	6115,4	39·20
4	57 49 35·92	9·927 5966,0	+13,0	0·13	+1,7	0	5966,0	35·92
	..	$\bar{1}$·506 1619,6	..	0·73	11,3	0,1	1619,7	..
	Right hand.							
1	53 43 28·06	9·906 4325,0	+16,0	0·15	+2,4	−0,8	4324,2	28·01
11	46 5 23·11	9·857 5900,2	+20,0	0·08	+1,6	−0,6	5899,6	23·08
16	75 19 35·47	9·985 5994,1	+ 5,5	0·08	+0,4	−0,1	5994,0	35·45
13	46 36 48·23	9·861 3762,6	+20,0	0·09	+1,8	−0,6	3762,0	48·20
3	51 46 11·76	9·895 1640,5	+16,5	0·10	+1,7	−0,6	1639,9	11·72
	..	$\bar{1}$·506 1622,4	..	0·50	7,9	−2,7	1619,7	..

Northern Pentagon.

	Left hand.							
11	46 5 23·11	9·857 5900,2	+20,0	0·08	+1,6	+0,4	5900,6	23·13
22	52 19 7·31	9·898 4087,0	+16,0	0·23	+3,7	+1,0	4088,0	7·37
24	56 55 42·53	9·923 2388,4	+14,0	0·08	+1,1	+0,3	2388,7	42·55
15	55 49 55·72	9·917 7133,1	+14,0	0·10	+1,4	+0,4	7133,5	55·75
8	52 59 45·33	9·902 3253,3	+16,0	0·06	+1,0	+0,2	3253,5	45·34
	..	$\bar{1}$·499 2762,0	..	0·55	8,8	+2,3	2764,3	
	Right hand.							
10	55 56 1·14	9·918 2346,0	+14,0	0·07	+1,0	−1,4	2344,6	1·04
21	47 17 26·53	9·866 1717,6	+20,0	0·14	+2,8	−4,0	1713,6	26·33
23	42 25 24·99	9·829 0505,8	+23,0	0·07	+1,6	−2,3	0503,5	24·89
14	51 40 39·20	9·894 6115,4	+17,0	0·04	+0,7	−1,0	6114,4	39·14
7	78 30 37·69	9·991 2088,8	+ 4,0	0·09	+0,4	−0,6	2088,2	37·55
	..	$\bar{1}$·499 2773,6	..	0·41	6,5	−9,3	2764,3	

TABLE III.—SHOWING THE APPLICATION OF THE THIRD CORRECTION, VIZ., THAT FOR COMBINING THE TWO PENTAGONS TO FORM A DOUBLE POLYGON.

1.	2.	3.	4.	5.	6.	7.	8.	9.
No.	Angles from Table II, Col. 9.	Logarithmic Sines.	*d*.	R.	*d* × R.	Correction to Logarithm.	Corrected Logarithm.	Corrected Angles.
	Left hand.							
	° ′ ″							″
2	57 27 44·07	9·925 8467,4	+13,5	0·27	3,6	−0,3	8467,1	44·04
12	44 6 37·59	9·842 6365,0	+22,0	0·14	3,1	−0,2	6364,8	37·58
17	55 23 59·26*	9·915 4706,8	+14,5	..	..	..	4706,8	59·26
14	51 40 39·17*	9·894 6114,9	+17,0	..	..	..	6114,9	39·17
4	57 49 35·92	9·927 5966,0	+13,0	0·13	1,7	−0,1	5965,9	35·92
	..	$\bar{1}$·506 1620·1	..	0·54	8,4	−0,6	1619,5	..
	Right hand.							
1	53 43 28·01	9·906 4324,2	+16,0	0·15	2,4	−0,4	4323,8	27·99
11	46 5 23·11*	9·857 5900,2	+20,0	..	..	..	5900,2	23·11
16	75 19 35·49*	9·985 5994,2	+ 5,5	..	..	..	5994,2	35·49
13	46 36 48·20	9·861 3762,0	+20,0	0·09	1,8	−0,3	3761,7	48·18
3	51 46 11·72	9·895 1639,9	+16·5	0·10	1,7	−0,3	1639,6	11·70
	..	$\bar{1}$·506 1620,5	..	0·34	5,9	−1,0	1619,5	..
	Left hand.							
11	46 5 23·11*	9·857 5900,2	+20,0	..	..	..	5900,2	23·11
22	52 19 7·37	9·898 4088,0	+16,0	0·23	3,7	−0,1	4087,9	7·37
24	56 55 42·55	9·923 2388,7	+14,0	0·08	1,1	..	2388,7	42·55
15	55 49 55·75	9·917 7133,5	+14,0	0·10	1,4	..	7133,5	55·75
8	52 59 45·34*	9·902 3253,5	+16,0	..	..	..	3253,5	45·34
	..	$\bar{1}$·499 2763,9	..	0·41	6,2	−0,1	2763,8	..
	Right hand.							
10	55 56 1·04	9·918 2344,6	+14,0	0·07	1,0	−0,2	2344,4	1·03
21	47 17 26·33	9·866 1713,6	+20,0	0·14	2,8	−0,7	1712,9	26·29
23	42 25 24·89	9·829 0503,5	+23,0	0·07	1,6	−0,4	0503,1	24·87
14	51 40 39·17*	9·894 6114,9	+17,0	..	..	..	6114,9	39·17
7	78 30 37·63*	9·991 2088,5	+ 4,0	..	..	..	2088,5	37·63
	..	$\bar{1}$·499 2765,1	..	0·28	5,4	−1,3	2763,8	..

* Means of the values found from Table II, column 9, and the Tables, p. 254.

TABLE IV.—COMPARISON OF ERRORS FOUND BY SHORT METHOD WITH THOSE OF THE G.T.S.I. (vol. ii. pp. 216, 217).

1.	2.	3.	4.	5.	6.	7.	8.
No.	Observed Angles.	Spheroidal Excess.	Plane Angles.	Corrected Plane Angles.	Error by Short Method.	Error by Rigorous Method.	Difference of Errors.
	″		″	″	″	″	″
1	28·63	0·50	28·13	27·99	+0·14	+0·1465	0·0065
2	43·19	0·44	42·75	44·04	−1·29	−1·3200	0·0300
3	11·71	0·44	11·27	11·70	−0·43	−0·4485	0·0185
4	36·55	0·57	35·98	35·92	+0·06	+0·0667	0·0067
5	4·42	0·44	3·98	4·26	−0·28	−0·2314	0·0486
6	54·90	0·50	54·40	54·43	−0·03	−0·0338	0·0038
7	38·05	0·50	37·55	37·63	−0·08	−0·0880	0·0080
8	46·09	0·44	45·65	45·34	+0·31	+0·2764	0·0336
9	36·44	0·56	35·88	35·90	−0·02	−0·0232	0·0032
10	2·18	0·82	1·36	1·03	+0·33	+0·3199	0·0101
11	23·49	0·50	22·99	23·11	−0·12	−0·0851	0·0349
12	38·15	0·50	37·65	37·58	+0·07	+0·0671	0·0029
13	48·84	0·57	48·27	48·18	+0·09	+0·0864	0·0036
14	39·86	0·44	39·42	39·17	+0·25	+0·2946	0·0446
15	56·50	0·56	55·94	55·75	+0·19	+0·2089	0·0189
16	36·34	0·44	35·90	35·49	+0·41	+0·3986	0·0114
17	59·48	0·50	58·98	59·26	−0·28	−0·3066	0·0266
18	52·50	0·83	51·67	51·61	+0·06	+0·0810	0·0210
19	51·68	0·77	50·91	51·15	−0·24	−0·1749	0·0651
20	40·17	0·57	39·60	39·39	+0·21	+0·1719	0·0381
21	27·26	0·77	26·49	26·29	+0·20	+0·1527	0·0473
22	8·83	0·82	8·01	7·37	+0·64	+0·6385	0·0015
23	25·70	0·56	25·14	24·87	+0·27	+0·2995	0·0295
24	43·28	0·77	42·51	42·55	−0·04	−0·0681	0·0281

Mean difference of errors = 0·0226.

(*Students' Paper No. 268.*)

"The Deflection of Spiral Springs."[1]

By Alfred E. Young, Stud. Inst. C.E.

The various formulas deduced from the theory of elasticity for the deflection of spiral springs, and in particular of conical spiral springs, are necessarily based upon certain assumptions as to the method of applying the load, the form of the section, &c.; and the object of the present Paper is to show how far the discrepancies between observation and experiment may be ascribed to the imperfect realization in practice of these assumed conditions.

The deflection of spiral springs is of peculiar interest, as a spiral spring under the action of a tensile or compressive force along the axis of the cone on which the spiral is described, is in the position of a beam experiencing all the four simple straining actions, namely, a twisting moment, a bending moment, a shearing force, and a pull or thrust. The effect of a load W at any point of the spiral can be found in the following way. Let there be introduced at the point A (*Figs. 1, 2*), two opposite forces, each equal to W, so that the equilibrium of the point will remain undisturbed:

Fig. 1.

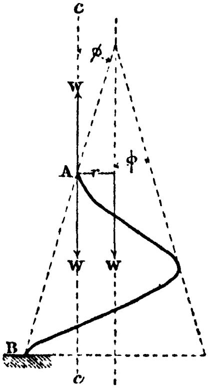
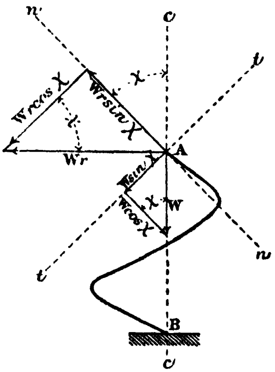


Fig. 2.



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[1] This communication was read and discussed at a meeting of the Students on the 28th of March, 1889, and was awarded a Miller Prize in the Session 1889-90.

then it is evident that the effect of W at A is equivalent to a couple Wr, together with the force W acting at A. Now the line of action of W is not parallel to a section normal to the centre line of the spiral at A, but makes an angle χ with the normal to such a section as shown in *Fig. 2.* It will, therefore, be convenient to resolve the couple and the force into couples and forces acting parallel and perpendicular to a normal section of the spiral. In *Fig. 2*, the couple Wr is represented by a line drawn perpendicular to its plane, and directed so that the turning appears clockwise when looking along the line in the direction of the arrow. This can be resolved into two components perpendicular to one another, one $= Wr \sin \chi$, tending to twist the spiral counter-clockwise, the other $= Wr \cos \chi$ tending to bend the spiral so as to diminish its radius of curvature. In the same manner W can be resolved into a shearing force $W \sin \chi$, and a thrust $W \cos \chi$. The effect of the latter two forces in deflecting the spiral is very small in comparison with that of the two former, and can always be neglected, and in most cases the angle χ is so nearly a right-angle that the bending moment is inappreciable, so that there only remains a twisting moment sensibly equal to Wr.

Circular Section.—In the Appendix it is shown that the deflection of a uniform conical spiral, under a load W is—

$$\delta = \frac{\pi C W}{2 \lambda G J} (R_n^4 - R_o^4), \text{ where}$$

C = a constant depending on the form of section = 1 in case of a circle;

λ = the increment of radius per coil;

G = the coefficient of transverse elasticity;

J = the polar moment of inertia of the section $= \frac{\pi d^4}{32}$ in the case of a circle, d being the diameter of the section.

R_n, R_o = the greatest and least radii of the spiral respectively. Inserting these values,

$$\delta = \frac{16 W}{\lambda G d^4} (R_n^4 - R_o^4).$$

The greatest stress f is given by $f = \frac{16 W R_n}{\pi d^3}$, and will therefore occur at the largest coil of the spring. If, however, the diameter of the wire decreases with the radius, that is to say, if $d \propto$

a spring of equal strength will be obtained. Writing $d^3 = \mathrm{K}\,\mathrm{R}$, where K is a constant, the deflection of the spring will be—

$$\frac{24\,\mathrm{W}}{\mathrm{G}\,\lambda\,\mathrm{K}^{\frac{4}{3}}}\left(\mathrm{R}_n^{\frac{8}{3}} - \mathrm{R}_o^{\frac{8}{3}}\right).$$

Putting f^1 for the proof stress of the material, we have $f^1 = \frac{16\,\mathrm{W}^1}{\pi\,\mathrm{K}}$, so that the greatest safe deflection is—

$$\delta^1 = \frac{1.5\,\pi\,f^1}{\mathrm{G}\,\lambda\,\mathrm{K}^{\frac{1}{3}}}\left(\mathrm{R}_n^{\frac{8}{3}} - \mathrm{R}_o^{\frac{8}{3}}\right),$$

and the greatest safe load—

$$\mathrm{W}^1 = \frac{\mathrm{K}\,\pi\,f^1}{16} = 0{\cdot}196\,f^1\,\mathrm{K}.$$

It may be remarked here that for a given length of spring and weight of material, the deflection is least when the section is circular. Moreover, if the material of the spring is tubular, of external radius R, and internal radius $c\mathrm{R}$, then the ratio of the stiffness to the weight is increased in the proportion of $1 + c^2 : 1$. The Author is not aware that any springs of this type have ever been constructed.

Elliptical Section.—This section is frequently met with in buffer springs, the larger diameter D of the section being in the vertical plane.

For an ellipse of larger diameter D, and smaller diameter d,

$$\mathrm{C} = \frac{(\mathrm{D}^2 + d^2)^2}{4\,\mathrm{D}^2\,d^2}, \quad \mathrm{J} = \frac{\pi\,\mathrm{D}\,d\,(\mathrm{D}^2 + d^2)}{64}.$$

Thus, if the section is uniform throughout,

$$\delta = \frac{8\,\mathrm{W}}{\lambda\,\mathrm{G}}\,\frac{(\mathrm{D}^2 + d^2)}{\mathrm{D}^3\,d^3}\left(\mathrm{R}_n^4 - \mathrm{R}_o^4\right);$$

or, if the coils just fit one another, $\lambda = d$, and then,

$$\delta = \frac{8\,\mathrm{W}}{\mathrm{G}}\,\frac{\mathrm{D}^2 + d^2}{\mathrm{D}^3\,d^4}\left(\mathrm{R}_n^4 - \mathrm{R}_o^4\right).$$

The greatest stress in the material is given by $f = \frac{16\,\mathrm{W}\,\mathrm{R}}{\pi\,\mathrm{D}\,d^2}$, and is found at the extremities of the shorter diameter. This is a particular case of the general result proved by St. Venant, that in beams subject to torsion, the surface stress is at a maximum at those points nearest the axis of torsion, and a minimum at those points farthest away from it.

In most cases it is usual to keep the dimension D constant, and make d diminish with the radius of the spring. If d^2 varies as R the maximum stress will be the same throughout. Let $d^2 = \mathrm{K\,R}$, and let J and C be expressed as functions of R, then—

$$\delta = \frac{64\,\mathrm{W}}{\mathrm{G\,D}\,\lambda\,\sqrt{\mathrm{K}}}\left\{\frac{1}{5\,\mathrm{K}}\left(\mathrm{R}_n^{\frac{5}{2}} - \mathrm{R}_o^{\frac{5}{2}}\right) + \frac{1}{7\,\mathrm{D}^2}\left(\mathrm{R}_n^{\frac{7}{2}} - \mathrm{R}_o^{\frac{7}{2}}\right)\right\}.$$

This is an awkward expression to work with, and in most cases it is preferable to substitute the mean value of d in the formula for the spring of uniform section.

Rectangular Section.—This is a very common form of section, especially in buffer-springs, &c., where it is customary to make the several coils of the spring overlap one another. This form is frequently termed a volute spring.

It was usually assumed that the moment of resistance to torsion of a rectangular beam was simply $\mathrm{GJ}\theta$, where θ is the angle of torsion per unit of length, and J the polar moment of inertia of the section. This is rigidly true for a circular section, but M. Barré de St. Venant showed that such is not the case for non-circular sections, the real moment of resistance being always less than that given by this formula. It is for this reason that the Author has introduced the factor C in the general formula of deflection, C being unity in the case of a circular section.

In the case of a rectangular beam, the value of C is determined by an infinite series involving the ratio of the sides of the rectangle. This series the Author has not thought it necessary to give, but he has calculated the value of it to a sufficient degree of accuracy for all ordinary purposes for a number of ratios of the sides starting with the square. The Author thinks that the use of these calculated coefficients will give more accurate results than the approximate formulas given in text-books for the resistance to torsion of rectangular beams.

If the larger side of the rectangle is denoted by b, and the smaller by c, then—

$$\begin{aligned} f_{\max} &= \alpha\,\mathrm{G}\,c\,\theta, \\ \mathrm{T} &= \beta\,\mathrm{G}\,b\,c^3\,\theta, \\ &= \gamma\,f\,b\,c^2, \end{aligned}$$

where α β and γ are the coefficients given in the Table. For a square beam, it will be seen that $\gamma = 0{\cdot}208$, so that $\mathrm{T} = 0{\cdot}208 f s^3$, where s is the side of the square. Rankine, probably by an oversight, gives the number 0·281 in his 'Applied Mechanics,' and this error has been copied again and again in other text-books. As may be seen from the Table, 0·281 is, in fact, almost exactly the value of γ for a rectangle of which one side is four times the length of the other.

TABLE OF COEFFICIENTS RELATIVE TO THE TORSION OF RECTANGULAR BEAMS.

Ratio of sides $\frac{b}{c} =$	$f_{\max} = \alpha G c \theta$ $\alpha =$	$T = \beta G \theta b c^3$ $\beta =$	$T = \gamma f b c^2$ $\gamma =$
1 (square)	0·675	0·141	0·208
2	0·930	0·229	0·246
3	0·985	0·263	0·267
4	0·997	0·280	0·282
5	0·999	0·291	0·291
6	1·000	0·298	0·298
10	1·000	0·312	0·312
20	1·000	0·323	0·323
100	1·000	0·331	0·331
∞	1·000	0·333	0·333

T = moment of resistance of section to torsion;

G = modulus of transverse elasticity;

θ = angle of torsion per unit length of beam, reckoned in radians $\left(1 \text{ radian} = \frac{180^\circ}{\pi}\right)$;

$f_{\max}$ = maximum stress in section, being the torsion shear at that point on the periphery of the section which is nearest the axis of torsion;

b and c = the sides of the section ($b > c$);

$$\theta = \frac{C\,T}{G\,J} = \frac{T}{\beta\, b\, c^3\, G}, \quad \therefore \frac{C}{J} = \frac{1}{\beta\, b\, c^3}.$$

The Table shows that when one side of the rectangle is more than four times the length of the other, the formula $f = G\theta c$ may be taken without sensible error, and when one side is more than ten times the other, $T = \frac{1}{3} f b c^2$, this being the value to which T continually approximates as the ratio of b to c increases.

Returning to the deflection of a spiral spring of rectangular section: for all ratios of sides commonly met with, $\frac{C}{J} = \frac{3}{b\,c^3}$. Substituting this in the general formula for a spring of uniform section,

$$\delta = \frac{3\,\pi\,W}{2\,\lambda\, G\, b\, c^3}\left(R_n^4 - R_o^4\right);$$

or, if the coils just fit one another, so that $\lambda = c$,

$$\delta = \frac{3\,\pi\,W}{2\,G\,b\,c^4}\,(R_n{}^4 - R_o{}^4).$$

Also, $f_{max} = \frac{3\,W\,R_n}{b\,c^2}$, so that tho greatest safe deflection is found to be—

$$\delta^1 = \frac{\pi\,f^1}{2\,G\,c^2}\;\frac{(R_n{}^4 - R_o}{R_n}$$

and the greatest safe load,

$$W^1 = \frac{b\,c^2\,f^1}{3\,R_n}.$$

In ordinary springs of this type it is customary to keep the depth of the strip b constant, and to make the thickness c diminish with the radius. It will be seen that if c^2 varies directly as R, the maximum stress will be everywhere the same. Putting, therefore, $c^2 = KR$, where K is a constant, and calculating the deflection after expressing C and J as functions of R, the expression becomes—

$$\delta = \frac{0{\cdot}479\,W}{G\,\lambda\,b\,K^{\frac{3}{2}}}\,(R_n{}^{\frac{5}{2}} - R_o{}^{\frac{5}{2}}).$$

In most cases, however, it will be sufficiently accurate to take the mean value of c, and use the formula for a spring of uniform section.

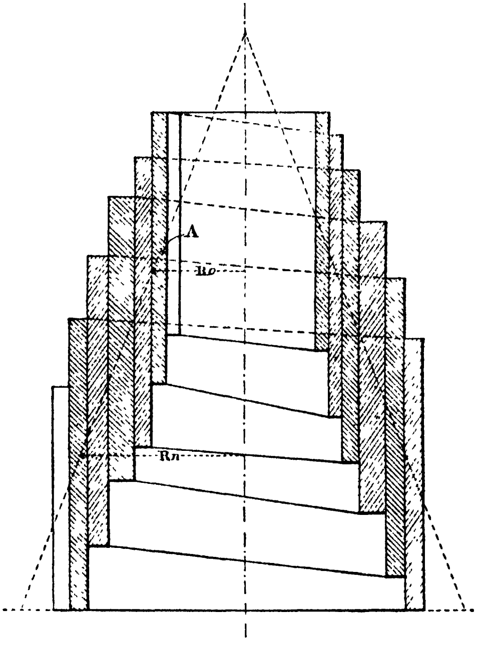
Square Section.—This is a special case of the latter section. The requisite formulas are given in the summary.

This completes the four ordinary forms of section. Other sections of various forms are sometimes met with. There may, of course, be practical reasons for departing from the circular form in certain cases, but so far as getting the stiffest spring from a given weight of material, and from a given length of strip is concerned, St. Venant's results prove that the circular section is the best, and that in all cases where the section has projecting ridges, there is a waste of material. Table I shows the ratios of the moment of resistance to torsion of a few sections to that of a circular section of the same area.

The formulas for cylindrical springs are given in the summary at the end of the Paper. It may be interesting to notice, however, that for a given weight of material and length of strip, a conical spiral spring is from three to four times as stiff as a cylindrical spiral having a radius equal to the largest radius of the conical spring. The conical spring has also the advantage that its range is larger than that of an equivalent cylindrical spring.

Through the kindness of Professor Unwin, the Author was enabled to make some experiments on a volute buffer spring in the laboratory of the City and Guilds Central Institution. A section of this spring, drawn one-third full size, is shown in *Fig. 3* It was made by Messrs. Turton and Sons, Sheffield, of cast-steel from Swedish iron, and the Author also procured four test specimens of the same material. Of these, two were hardened and two unhardened—one hardened and one unhardened piece being tension specimens, and the other two pieces torsion specimens. The object of the experiments on these test-pieces was to find the modulus of elasticity, both direct and transverse, the limit of elasticity both in tension and torsion, the breaking stress in either case, and the effect of hardening on all these quantities (Table II).

Fig. 3.



Scale ⅓ full size.

Section of Buffer-Spring Experimented on.

The modulus of direct elasticity E, was measured on the tension specimens, using Professor Unwin's mirror apparatus.[1] It appears from the results of the experiments given in the annexed Tables that hardening does not affect the value of E, the modulus being the same for each test-bar within the limits of experimental error —namely, 13,100 tons per square inch. The stress at elastic limit was much higher in the case of the hardened specimen than in the case of the unhardened one, being about 67 tons per square inch for the former, and about 44 tons per square inch for the latter. Both specimens broke with sensibly the same stress, namely, about 68 tons per square inch, showing that the hardened specimen was elastic almost up to the instant of fracture.

The modulus of transverse elasticity G, was measured by observing the angle of torsion on a 5-inch length of the bar, when it was subjected to a known twisting moment. The experiments show that the modulus of transverse elasticity is rather higher in the hardened than in the unhardened bar, namely, about 5,600 tons per square inch for the former, and 5,200 tons per square inch for the latter. The stress at elastic limit for the hardened bar was not ascertained; for the unhardened bar it was about 21 tons per square inch. The hardened bar broke with a calculated stress of 73·7 tons per square inch of shear at the surface, while, for the unhardened bar the stress at rupture was 50·7 tons per square inch. These high values are no doubt due to the false assumption that the material is elastic up to fracture (see Unwin's "Testing Materials of Construction," p. 333).

Before discussing the experiments made on the spring itself, it may be noted that the formula for the deflection is calculated on the assumption that the load is truly axial, that is to say, a concentrated load passing through the axis of the cone on which the centre line of the spring lies, and that the deflections are measured from the free end to the fixed end of the spring, also on the assumption that this last is actually a fixed point. But in practice, the top and bottom coils of the spring are cut off parallel to one another, and perpendicular to the axis of the cone, so that the bottom coil gives a fair bearing-surface to rest upon the support, and the top one a fair bearing surface for the load, which is generally applied by means of a plate resting on the top coil. Thus, instead of the load being a concentrated one, it is distributed over the edge of the top coil. But in considering the effect of this load on any part of the spring other than the top coil, the resultant, which is equivalent to a concentrated load acting through the axis of the cone, may be taken. And since the top coil always remains in contact with the load, it does not move in relation to the load, and therefore takes no share of the deflection. In applying the formula, therefore, R_o must not be taken as the radius of the top coil, but as the radius at that part of the spring where it just leaves the bearing surface on which the load rests. Similar reasoning applies to the bottom coil. This takes no share of the deflection of the spring, but merely acts as a support, and R_n must be measured from that part of the spring which just leaves the bottom bearing surface.

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[1] Proceedings of the Physical Society, vol. viii. p. 178; Minutes of Proceedings Inst. C.E., vol. lxxxviii. p. 92.

It will be observed from the drawing that the spring experimented on had six complete coils, but from what has just been said only four of these were available for deflection. The depth of the section was uniform, and the thickness varied in a rather peculiar way, being least at the first coil, then increasing up to the end of the third coil, and after that diminishing again. The total length of the uncoiled strip was about 70 inches, and the weight about 23 lbs. Taking the mean dimensions of the spring, and applying the formula for a conical spiral of uniform section, the deflection with a load of 0·2 ton is—

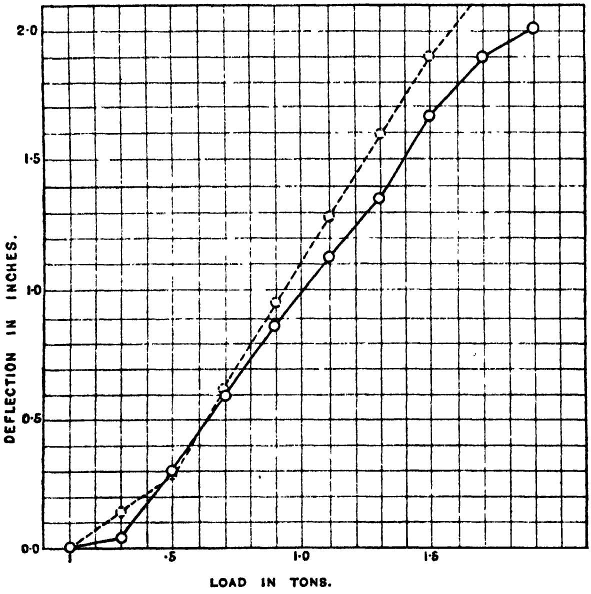
$$\frac{3\,\pi W\,(R_n^4 - R_o^4)}{2\,G\,\lambda\,b\,c^3} = 0{\cdot}276 \text{ inch nearly.}$$

$R_n = 2{\cdot}62$ inches $R_n^4 - R_o^4 = 43{\cdot}27$;
$R_o = 1{\cdot}40$ „ $G = 5600$;
$b = 4{\cdot}5$ „ $\lambda = 0{\cdot}3$ inch;
$c = 0{\cdot}269$ inch (allowing $\frac{1}{32}$ inch for clearance).

In carrying out the experiments, the spring was compressed between the compression-tables of the 100-ton testing-machine at the Central Institution, and the distance between the tables was measured with a vernier scale reading to $\frac{1}{1000}$ of an inch, for each increment of 0·2 ton load, starting with a zero load of 0·1 ton, and usually stopping with a load of 1·1 ton. At first the readings were very irregular, owing apparently to friction, and the clogging up of the coils with paint. After a time, by scraping and oiling the spring, the grosser irregularities were got rid of, and more consistent results were obtained. The annexed Table shows the results of one of the later experiments; these are also given by the black line in *Fig. 4*.

The diagram (*Fig. 4*) shows that the spring is much stiffer for the first two or three loads than for the next five or six, and after that becomes stiffer again. This is the character of all the diagrams for this spring. The Author was for a long while puzzled to account for the small deflections with the first two or three loads, and various devices were tried to get rid of this discrepancy. It must be caused either by friction among the coils, which, as the load increases, becomes less sensible; or by the load not being truly axial at first, an examination of the spring, showing that the top coil was cut off not quite parallel to the bottom one. To eliminate this latter source of error, the top coil was filed off quite parallel, and at Professor Unwin's suggestion, a wooden block, with the upper surface turned spherical, was fitted into the top coil, so that the load, being in contact with the spherical surface of the wood, must pass through the axis of the spring, and the results obtained in this way are shown by the dotted line in *Fig. 4*. It will be seen that the discrepancies are still observable, the spring being stiffer till a deflection of about 0·29 inch is reached, after which the deflection for each increment

Fig. 4.



Deflection Diagram for Buffer-Spring.

of load is regular and in accordance with calculation up to 1·9 inch, when it falls off again. The deflection for 0·2 ton is, however, greater when the spherical plug is used than without it, as might have been expected.

With regard to the deflections falling off again when the higher loads are reached, the following seems to be the explanation:—The spring is compressed top and bottom, between two plane surfaces, the top and bottom coils respectively being in contact with these surfaces all round their edge. As the load increases, the lower edge of the bottom coil but one gradually approaches the table of the machine, until, when a certain load is reached, the whole of that coil is in contact with the lower table, so that for the purposes of deflection the spring has lost the effect of that coil, and now is virtually a spring of three coils only, and in calculat-

TABLES OF RESULTS OF EXPERIMENTS ON VOLUTE BUFFER-SPRING.

(Black line in diagram.)

Load.	Vernier Readings.	Differences.	Total Deflection.
Tons.		Inches.	Inches.
0·1	1·505	..	..
0·3	1·472	0·033	0·033
0·5	1·208	0·264	0·297
0·7	0·916	0·292	0·589
0·9	0·645	0·271	0·860
1·1	1·433 1·172	0·261	1·121
1·3	0·943	0·229	1·350
1·5	0·477 0·177	0·300	1·650
1·7	1·148	0·250	1·900
1·9	0·898 2·462 2·349	0·113	2·013

USING SPHERICAL SEATING.

(Dotted line in diagram.)

0·1	2·753	..	..
0·3	2·610	0·143	0·143
0·5	2·466	0·144	0·287
0·7	2·131	0·335	0·622
0·9	1·805	0·326	0·948
1·1	1·472	0·333	1·280
1·3	1·160	0·312	1·593
1·5	0·848	0·312	1·905
1·7	0·575	0·273	2·178

ing any further deflection R_n must be measured to the end of the fourth coil. Similar reasoning applies to the top coils, which gradually approach the surface of the upper table. As, however, the load has a greater leverage on the lower coils, the spring closes up from the bottom and not from the top. Observation shows this to be the case.

As a check on the above, the load required to close the fifth, or bottom coil but one, can be calculated from the formula. The pitch is about 1 inch. Hence, inserting the proper values—

$$W = \frac{2\,\delta\,G\,\lambda\,b\,c^3}{3\,\pi\,(R_n^4 - R_o^4)} = 1\cdot 7 \text{ ton.}$$

This is just about the value of the load at which the deflections actually fell off. The load required to close up the Author's spring was 4 tons. It does not appear that in this spring the increased stiffness with higher loads was wholly due to the action just explained. A great part of it was the result of friction among the top coils, which, as the bottom coils came out of action, made itself more apparent. An examination of the spring showed that there was much more clearance among the lower than among the upper coils, the second and third coils bearing tightly together at one part. This view is confirmed when the intensity of stress with the various loads is calculated, and this can be done in two ways, either from the deflection or from the load. Usually stresses are calculated from the forces acting, and not from the deflection produced, because most often the forces are more accurately known than the deflections; but in a case like this, in which it is not known how much of the load is required for overcoming friction, more accurate values may be obtained by calculating from the deflection.

In the formula given in the preceding part of the Paper for the torsion of rectangular beams, it is shown that the maximum stress, when calculated from the deflection, is—

$$a\,G\,c\,\theta;$$

where $a = 1$ in this case, since $b = 20\,c$ nearly;
G = modulus of transverse elasticity;
θ = angle of torsion per unit length of beam.

In applying this to a spiral spring, θ may be taken as the deflection divided by the mean radius and by the length of the strip.

Then the maximum stress in the fifth coil, just before it closes, is—

$$f_{\max} = \mathrm{G}\, c\, \theta = \frac{\mathrm{G}\, c\, p}{2\, \pi\, \mathrm{R}^2} = 40 \text{ tons per square inch;}$$

where p = pitch = 1 inch;
R = mean radius = 2·45 inches;
c = 0·27 inch.

In the same way the stress in the second coil when it just closes is—

Deflection = 0·6 inch; c = 0·25 inch;
R = mean radius = 1·5 inch;

$$f_{\max} = \frac{5{,}600 \times 0{\cdot}25 \times 0{\cdot}6}{2 \times \pi \times 1{\cdot}5 \times 1{\cdot}5} = 59 \text{ tons per square inch.}$$

Applying the formula which gives the stress in terms of the load, namely—

$$f_{\max} = \frac{3\, \mathrm{W}\, \mathrm{R}_m}{b\, c^2},$$

in the first case—

$$\frac{3 \times 1{\cdot}7 \times 2{\cdot}45}{4{\cdot}5 \times 0{\cdot}27^2} = 38 \text{ tons per square inch;}$$

and for the maximum stress in second coil when just closed—

$$f_{\max} = \frac{3 \times 4 \times 1{\cdot}5}{4{\cdot}37 \times 0{\cdot}25^2} = 66 \text{ tons per square inch.}$$

That is, 7 tons per square inch more than when calculated from the deflection, which shows conclusively that there must have been a considerable amount of friction between the second and third coils.

The above figures show what high values the stresses in buffer-springs of this type may reach. These stresses are the torsional shears at the middle points of the longer sides of the section, namely, the side b. In the table of experiments, the hardened bar broke with a calculated stress of this kind of 73·7 tons per square inch, and from the above figures it would appear that this cannot have been very far from the truth, the plastic yielding of the bars being very small, as might be inferred from the stress-strain diagram obtained in the tension test. These high values are to the same extent borne out by the experiments of Mr. Hartnell, on governor and valve springs, who finds such springs safe with from 25 to 30 tons per square inch for ⅜-inch wire, and 20 tons per square inch for ½-inch wire.

Mr. Hartnell says:[1] "The size of a spiral spring may be calculated from the formula (p. 304) of 'Rankine's Useful Rules and Tables,' but experience with Salter's springs has shown that the safe limit of stress is more than twice as great as there given,[2] namely, 60,000 to 70,000 lbs. per square inch of section, with ⅜-inch to ¼-inch wire, and about 50,000 with ½-inch wire. Hence the work that can be done by springs of wire is four or five times as great as Rankine allows.

For ⅜-inch wire and under, the maximum load in lbs.,

$$= \frac{12{,}000 \times (\text{diameter of wire})^3}{\text{Mean radius of springs}}.$$

Weight in lbs. to deflect spring 1 inch

$$= \frac{180{,}000 \times (\text{diam.})^4}{\text{Number of coils} \times (\text{rad.})^3}.$$

"The work in foot-pounds that can be stored up in a spiral spring would lift it above 50 feet.

"In a few rough experiments made with Salter's springs, the coefficient of rigidity (G) was noticed to be 12,600,000, (5,630) to 13,700,000, (6,120) with ¼-inch wire; 11,000,000, for $\frac{11}{32}$-inch; and 10,600,000 to 10,900,000 for ⅜-inch wire."

These experiments show that a very fair estimate of the actual behaviour of spiral springs may be obtained from the theoretical formulas. The discrepancies due to friction among the coils would no doubt disappear with wear, the spring experimented on being a quite new one; and the deflection line when friction is eliminated would be a straight line starting from the origin, and then gradually becoming concave towards the axis of loads, from the deflection at which the last coil but one comes out of action until the whole spring is closed up. The reason why a volute spring was chosen was because the Author found that considerable difficulty had hitherto been experienced in getting a straight deflection line with such springs, and he wanted to see how far the discrepancies might be accounted for by careful observation of what goes on during the action of the load on the spring, and to what extent they might be eliminated by complying as nearly as possible with the assumptions made in deducing the formulas.

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[1] Note B of Paper by W. Hartnell on "Automatic Expansion-Gear," Proc. Inst. Mech. Eng., Aug. 1882.

[2] Rankine gives 30,000 lbs. per square inch = 13·4 tons per square inch.

Before concluding the Author wishes to express his indebtedness to Professor Unwin, for permission to make the experiments at the Central Institution, and for many valuable suggestions in carrying them out; also to Mr. J. Taylor, laboratory assistant, and all those of his fellow-students who assisted him in the experiments.

The Paper is illustrated by several diagrams from which the *Figs.* in the text have been engraved.

APPENDIX.

TABLE I.—RELATIVE VALUES OF VARIOUS SECTIONS FOR RESISTING TORSION TO THAT OF A CIRCULAR SECTION OF EQUAL AREA.

Form of Section.		Relative Value.
Ellipse	$D = 2d$	0·80
Rectangle	$b = 10c$	0·21
Square		0·88
Square, with slightly hollow sides and rounded corners		0·87
Triangle (equilateral)		0·73

TABLE II.—TENSION EXPERIMENTS.

Dimensions of specimens, 1·95 inch × 0·282 inch.
Extensions measured with mirror apparatus on 8·1 inches.
" " " automatic " 10·0 "

Specimen.	Stress at Elastic Limit.	Stress at Rupture.	Elongation on 10-inch.	Modulus of Direct Elasticity E.
	Tons per square inch.	Tons per square inch.	Per cent.	Tons per square inch.
Hardened Swedish spring steel	67·73	67·33	..	13,110
Unhardened Swedish spring steel	43·6	68·77	4·8	13,130

$$E = \frac{\text{stress} \times \text{original length}}{\text{extension}}.$$

TORSION EXPERIMENTS.

Dimensions of specimens, 0·629 inch diameter.
Angle of torsion measured on 5-inch length.

Specimen.	Stress at Elastic Limit.	Calculated Stress at Rupture.	Modulus of Transverse Elasticity G.
	Tons per square inch.	Tons per square inch.	Tons per square inch.
Hardened Swedish spring steel	Not ascertained	73·7	5,600
Unhardened Swedish spring steel	21·0?	50·7	5,200

$$\frac{\text{twisting moment} \times \text{length}}{\text{polar moment of inertia} \times \text{observed angle of torsion}}.$$

APPENDIX.—SUMMARY OF FORMULAS FOR THE DEFLECTION OF SPIRAL SPRINGS.

Form of Section.	CONICAL SPIRAL.		CYLINDRICAL SPIRAL.		
	Deflection.	Maximum Stress.	Deflection.	Maximum Stress.	Rotation of Spring.
	Inches.	Tons per Square Inch.	Inches.	Tons per Square Inch.	Degrees.
Circular.	$\frac{16\,W}{\lambda\,G\,d^4}(R_n{}^4 - R_o{}^4)$	$\frac{16\,W\,R_n}{\pi\,d^3}$	$\frac{64\,n\,W\,R^3}{G\,d^4}$	$\frac{16\,W\,R}{\pi\,d^3}$	$\frac{360\,n\,W\,R^2}{\pi\,d^4}\left(\frac{64}{E} - \frac{32}{G}\right)\cos\chi$ $\cos\chi = \frac{p}{\sqrt{p^2 + 4\,\pi^2\,R^2}}$
Elliptical.	$\frac{8\,W\,(D^2+d^2)}{\lambda\,G\,D^3\,d^3}(R_n{}^4 - R_o{}^4)$	$\frac{16\,W\,R_n}{\pi\,D\,d^2}$	$\frac{32\,n\,W\,R^3}{G\,D^3\,d^3}(D^2 + d^2)$	$\frac{16\,W\,R}{\pi\,D\,d^2}$	$\frac{360\,n\,W\,R^2}{\pi\,D\,d^3}\left(\frac{64}{E} - \frac{16(D^2+d^2)}{G\,D^2}\right)\cos\chi$
Rectangular. $b > 6\,c$	$\frac{3\,\pi\,W}{2\,\lambda\,G\,b\,c^3}(R_n{}^4 - R_o{}^4)$	$\frac{3\,W\,R_n}{b\,c^2}$	$\frac{6\,n\,\pi\,W\,R^3}{G\,b\,c^3}$	$\frac{3\,W\,R}{b\,c^2}$	$\frac{360\,n\,W\,R^2}{b\,c^3}\left(\frac{12}{E} - \frac{3}{G}\right)\cos\chi$
Square.	$\frac{3{\cdot}54\,\pi\,W}{\lambda\,G\,s^4}(R_n{}^4 - R_o{}^4)$	$\frac{4{\cdot}79\,W\,R_n}{s^3}$	$\frac{9{\cdot}58\,n\,\pi\,W\,R^3}{G\,s^4}$	$\frac{4{\cdot}79\,W\,R}{s^3}$	$\frac{360\,n\,W\,R^2}{s^4}\left(\frac{12}{E} - \frac{7{\cdot}09}{G}\right)\cos\chi$

The letter n refers to the number of coils in the cylindrical spiral spring, and p denotes the pitch.

By "Rotation of the Spring" is meant the angle through which the free end turns under the action of the load, in a plane perpendicular to the axis of the spiral. This property of spiral springs has been made use of by Messrs. Ayrton and Perry in their patent magnifying spring as a means of measuring the very small extensions of the spring produced by electrical forces.

THEORY OF A CONICAL SPIRAL SPRING.

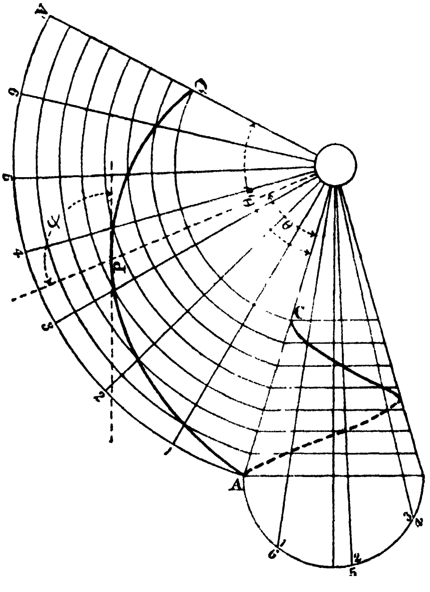
In *Fig. 5* the curve AC^1 is the development of the spiral AC. Let the distance AC, which is the pitch, be denoted by p, and the distance OA by ρ_n, where n represents the number of turns. Then if ρ is any radius vector through P, we have—

$$\rho = \rho_n - p\,\frac{\theta}{\theta_n},$$

where θ is the angle AOP, and θ_n the angle AOA^1, which is also $= 2\,\pi \sin \phi$, ϕ being the semi-vertical angle of the cone. Let r be the radius of the spiral at P, then $r = \rho \sin \phi$, and hence—

$$r = \frac{2_\pi\, r_n - p\,\theta}{2_\pi}.$$

Fig. 5.



Denoting the angle between ρ and the tangent at P by ψ, the following relations hold:—

$$\sin^2 \psi = \frac{(p\,\theta - 2_\pi\, r_n)^2}{p^2 + (p\,\theta - 2_\pi\, r_n)^2};\ \cos^2 \psi = \frac{p^2}{p^2 + (p\,\theta - 2_\pi\, r_n)^2};$$

and—

$$d\,s = \sqrt{\rho^2 + \left(\frac{d\,\rho}{d\,\theta}\right)^2}\,d\,\theta \;=\; \sqrt{\frac{p^2 + (2_\pi\, r_n - p\,\theta)^2}{2_\pi \sin \phi}}\,d\,\theta.$$

If the angle between the tangent at P and a vertical drawn through P be denoted by χ, it is known that—

$$\cos^2 \chi = \cos^2 \phi \cos^2 \psi,$$
$$\sin^2 \chi = 1 - \cos^2 \phi \cos^2 \psi.$$

Now let A B in *Fig. 1* represent a portion of the centre line of a conical spiral spring, the end B being considered as fixed. Let this spring be subjected to an axial load W. Then, following Poinsot's method, this force W may be transferred to any part of the spring A B, provided there is introduced a couple of transference = Wr, where r is the radius of the spring at the point considered.

Fig. 2 shows A B turned through a right angle; $n\,n$ is a section of the spring normal to its centre line, and $t\,t$ is the direction of the tangent to the centre line at A. The load W, acting at A, can therefore be resolved into a shearing force = W sin χ, and a normal force = W cos χ. Also the couple of transference Wr can be resolved into a twisting moment = Wr sin χ, and a bending moment = Wr cos χ.

Now the work done on an elastic beam under the action of twisting and bending moments, and normal and shearing forces is—

$$U = \tfrac{1}{2}\int_{B}^{A}\left[\frac{P^2}{E\,\Omega} + \frac{A\,S^2}{G\,\Omega} + \frac{M^2}{E\,I} + \frac{C\,T^2}{G\,J}\right] d\,s,$$

taken over the whole length of the beam, where—

P = normal force at any section;
S = shearing ,, ,, ,,
M = bending moment ,, ,,
T = twisting ,, ,, ,,
E = Young's modulus;
G = coefficient of transverse elasticity;
Ω = area of section;
I = moment of inertia of section about axis of flexure;
J = polar moment of inertia of section;
A and C constants depending on form of section.

In the present case—

$$U = \frac{W^2}{2}\int_{B}^{A}\left[\frac{\cos^2\chi}{E\,I} + \frac{A\sin^2\chi}{G\,\Omega} + \frac{r^2\cos^2\chi}{E\,I} + \frac{C\,r^2\sin^2\chi}{G\,J}\right] d\,s,$$

and the deflection of the spring in the direction of W—

$$\frac{d\,U}{d\,W} = W\int_{B}^{A}\left[\frac{\cos^2\chi}{E\,I} + \frac{A\sin^2\chi}{G\,\Omega} + \frac{r\cos^2\chi}{E\,I} + \frac{C\,r^2\sin^2\chi}{G\,J}\right] d\,s.$$

For practical purposes, the part of the deflection due to the first two terms may be neglected, and if the inclination of the spirals to the horizontal is small, the third term may also be neglected, so that the deflection in ordinary spiral springs is mainly due to the twisting moment—a result pointed out by Binet in 1814.

Spiral of Uniform Section.

In this case C and J are constants, and—

$$\delta = \frac{C\,W}{G\,J}\int_{B}^{A} r^2 \sin^2\chi\; d\,s,$$

or if bending is taken into account—

$$\delta = \frac{C\,W}{G\,J}\int_{B}^{A} r^2 \sin^2\chi\, d\,s + \frac{W}{E\,I}\int_{B}^{A} r^2 \cos^2\chi\, d\,s$$

$$= \frac{C\,W}{G\,J}\int_{B}^{A} r^2\, d\,s + W\left(\frac{1}{E\,I} - \frac{C}{G\,J}\right)\int_{B}^{A} r^2 \cos^2\chi\, d\,s$$

$$= \frac{C\,W}{G\,J}\int_{B}^{A} r^2\, d\,s + W\cos^2\phi\left(\frac{1}{E\,I} - \frac{C}{G\,J}\right)\int_{B}^{A} r^2 \cos^2\psi\, d\,s.$$

Expressing r, $d\,s$, and $\cos^2\psi$ as functions of θ, the deflection of one end of a complete spiral in relation to the other is—

$$\frac{C\,W}{8\,\pi^3 \sin\phi}\int_{o}^{2\pi\sin\phi} (2\,\pi\, r_n - p\,\theta)^2 \sqrt{p^2 + (2\,\pi\, r_n - p\,\theta)^2}\, d\,\theta$$

$$+ W\cos^2\phi\left(\frac{1}{E\,I} - \frac{C}{G\,J}\right)\int_{o}^{2\pi\sin\phi} \frac{p^2\,(2\,\pi\, r_n - p\,\theta)^2}{\sqrt{p^2 + (2\,\pi\, r_n - p\,\theta)^2}}\, d\,\theta$$

Integrating, and putting $p \sin\phi = \lambda$ the increment of radius per coil—

$$\frac{C\,W}{32\,\pi^3\,\lambda\, G\,J}\Bigg[\,2\,\pi\left\{r_n\,(p^2 + 4\,\pi^2\, r_n^{\,2})^{\frac{3}{2}} - r_{n-1}\,(p^2 + 4\,\pi^2\, r^2_{n-1})^{\frac{3}{2}}\right\}$$

$$- \frac{p^2}{2}\Big\{2\,\pi\, r_n \sqrt{p^2 + 4\,\pi^2\, r_n^{\,2}} - 2\,\pi\, r_{n-1}\sqrt{p^2 + 4\,\pi^2\, r^2_{n-1}}$$

$$+ p^2 \log \frac{2\,\pi\, r_n + \sqrt{p^2 + 4\,\pi^2\, r_n^{\,2}}}{2\pi\, r_{n-1} + \sqrt{p^2 + 4\pi^2\, r^2_{n-1}}}\Bigg] + \frac{W\cos^2\phi\, p^2}{16\,\pi^3\,\lambda}\left(\frac{I}{E\,I} - \frac{C}{G\,J}\right)\Bigg[\,2\,\pi\Big\{r_n\sqrt{p^2 + 4\pi^2\, r^2_n}$$

$$- r_{n-1}\sqrt{p^2 + 4\,\pi^2\, r^2_{n-1}}\Big\} - p^2 \log \frac{2\,\pi\, r_n + \sqrt{p^2 + 4\,\pi^2\, r^2_n}}{2\,\pi\, r_{n-1} + \sqrt{p^2 + 4\,\pi^2\, r^2_{n-1}}}\Bigg].$$

Fortunately in most cases p^2 may be neglected in comparison with $4\,\pi^2\, r_n^{\,2}$, and then taking twisting only into account—

$$\delta = \frac{\pi\, C\, W}{2\,\lambda\, G\,J}\,(r_n^{\,4} - r_{n-1}^{\,4}) \text{ approximately.}$$

To obtain the deflection of the whole spring, the deflections of all the separate coils must be added together from R_o the least radius, to R_n the greatest. Thus for the whole spring—

$$\delta = \frac{\pi\, C\, W}{2\,\lambda\, G\,J}\,(R_n^{\,4} - R_o^{\,4}).$$

Putting $\lambda = o$ in the general expression, so that all the coils have the same radius, and evaluating the indeterminate forms which occur, the formula for a cylindrical spiral is obtained:—

$$\delta = \frac{C\,W}{G\,J}\, r^2 \sqrt{p^2 + 4\,\pi^2\, r^2} + W\left(\frac{1}{E\,I} - \frac{C}{G\,J}\right)\frac{r^2\, p^2}{\sqrt{p^2 + 4\,\pi^2\, r^2}},$$

or if the pitch be small—

$$\delta = \frac{2\,\pi\, C\, W\, r^3}{G\,J}.$$

for each coil, so that if there are n coils—

$$\delta = \frac{2 n \pi C W r^3}{G J}.$$

When the spiral is not of uniform section, the quantities C and J will be variables, and must be put under the sign of integration, after being expressed as functions of θ.

Work Stored in the Spring.

The work stored is the work done in deforming the spring

$$= \int_B^A \delta \, d W$$

$$= \frac{W^2 \pi C}{4 \lambda G J} (R_n^4 - R_o^4)$$

for a conical spiral.

$$= \frac{W^2 n \pi C R^3}{G J}$$

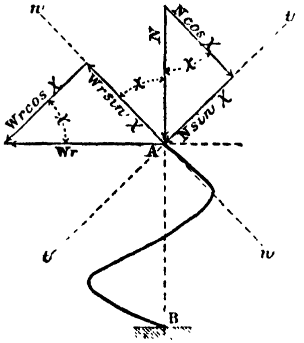
for a cylindrical spiral.

Rotation of the Spring.

In some springs the rotation of one end of the spiral in relation to the other under the action of an extending or compressing force is made use of.

The amount of rotation may be determined in the following way. Let it be supposed that, in addition to the load W, there is applied to the free end of the spring a right-handed couple N, acting in a plane normal to the axis of the spiral. Then, as shown in *Fig. 6*, N can be resolved into a twisting moment $- N \cos \chi$ and a bending moment $N \sin \chi$. Thus the total twisting moment will now be—

Fig. 6.



$$W r \sin \chi - N \cos \chi;$$

and the total bending moment—

$$W r \cos \chi + N \sin \chi.$$

Substituting these values in the formula for the work done on the spring—

$$U = \tfrac{1}{2}\int\left[\frac{C\,(W\,r\sin\chi - N\cos\chi)^2}{G\,J} + \frac{(W\,r\cos\chi + N\sin\chi)^2}{E\,I}\right] d\,s$$

taken over every coil.

Differentiating this with respect to N, the angle through which the end A rotates with respect to B, or $\frac{d\,U}{d\,N} =$

$$\omega = \int_B^A\left[\frac{(N\cos^2\chi - W\,r\sin\chi\cos\chi)}{G\,J} + \frac{(N\sin^2\chi + W\,r\sin\chi\cos\chi)}{E\,I}\right] d\,s.$$

Putting $N = o$ this formula gives the rotation under the action of W alone—

$$\omega = W\int_B^A r\sin\chi\cos\chi\,\frac{1}{E} - \frac{C}{G\,J}\Big)\,d\,s.$$

According as this is positive or negative, the rotation will be in the direction assumed for N or the contrary direction.

For a conical spiral—

$$\omega = \frac{W\,(R_n^2 - R_o^2)}{2\tan\phi}\left[\frac{1}{E\,I} - \frac{C}{G\,J}\right] \text{ radians}$$

$$= \frac{90\,W\,(R_n^2 - R_o^2)}{\pi\tan\phi}\left[\frac{1}{E\,I} - \frac{C}{G\,J}\right] \text{ degrees.}$$

For a cylindrical spiral—

$$\omega = 2\,n\,\pi\,W\,r^2\left(\frac{1}{E\,I} - \frac{C}{G\,J}\right)\cos\chi \text{ radians}$$

$$= 360\,n W\,r^2\left(\frac{1}{E\,I} - \frac{C}{G\,J}\right)\cos\chi \text{ degrees.}$$

Length of the Spring.

For a conical spiral—

$$L = \int_B^A\frac{\sqrt{p^2 + (2\,\pi\,r_n - p\,\theta)}}{2\,\pi\sin\phi}\,d\,\theta$$

$$= \frac{\pi}{\lambda}\,(R_n^2 - R_o^2) = n\,\pi\,(R_n + R_o) \text{ nearly.}$$

For a cylindrical spiral—

$$L = n\sqrt{p^2 + 4\,\pi^2\,R^2} = 2\,n\,\pi\,R \text{ nearly.}$$

Twist in the Spring.

For a conical spiral—

$$\theta = \frac{d\,U}{d\,T} = W\int_B^A\frac{C\,r\sin^2\chi}{G\,J}\,d\,s =$$

$$\frac{2\,\pi\,C\,W}{3\,\lambda\,G\,J}\,(R_n^3 - R_o^3) \text{ radians}$$

$$= \frac{120\,C\,W}{\lambda\,G\,J}\,(R_n^3 - R_o^3) \text{ degrees.}$$

For a cylindrical spiral—

$$\theta = \frac{2\,n\,\pi\,W\,R^2\,C}{G\,J} \text{ radians}$$

$$= \frac{360\,n\,W\,R^2\,C}{G\,J} \text{ degrees.}$$

Maximum Stress in Rectangular Section.

Let b and c be the sides of the section of which b is greater than c, then according to St. Venant—

$$f_{\max} = G\,c\,\theta \left[1 - \frac{16}{\pi^2} \sum_{n=1}^{n=\infty} \frac{1}{(2n-1)^2 \left(e^{\frac{(2n-1)\pi b}{2c}} + e^{-\frac{(2n-1)\pi b}{2c}}\right)}\right].$$

where e is the base of Napierean logarithms.

The moment of resistance to torsion of the section is—

$$T_1 = G\,\theta\,b\,c^3 \left[\frac{1}{3} - \frac{64\,c}{\pi^5\,b} \sum_{n=1}^{n=\infty} \frac{\dfrac{\left(e^{\frac{(2n-1)\pi b}{2c}} - e^{-\frac{(2n-1)\pi b}{2c}}\right)}{e^{\frac{(2n-1)\pi b}{2c}} + e^{-\frac{(2n-1)\pi b}{2c}}}}{(2n-1)^5}\right].$$

The quantity within the square brackets is also equal to the coefficient β in the paper

$$\beta = \frac{J}{C\,b\,c^3}.$$

OBITUARY.

General EDWARD FROME, late Colonel Commandant Royal Engineers, was born on the 7th of January, 1802. He died on the 12th of February, 1890. As a Cadet at Woolwich he studied engineering under Professors Olinthus Gregory and Peter Barlow, and attended the science lectures of Michael Faraday. In all his examinations while there, and for his commission in the Royal Engineers, he took the first place.

In 1827 as a First Lieutenant of Royal Engineers he was sent to Canada, where he made the whole of the surveys for the Rideau Canal from Ottawa to Kingston, and superintended the work or the main part of it, from its commencement to its completion.

In 1833 he was appointed Instructor of Surveying and Engineering at Chatham, and published the first edition of his work on engineering, which has since reached its fourth edition, and is a standard work. In 1839 he was appointed Surveyor-General of South Australia, and accepted the office on the condition that he was to have sole control of his work and not be as regarded it in any way subject to receive instructions from the Governor. The land affairs of the Colony were at that time in a state of chaos, such surveys as had been made being isolated and grossly inaccurate, and having cost on an average 2*s.* 6*d.* per acre.

Captain Frome, in a comparatively short time, brought the work, which had been so seriously in arrear as to place the authorities in most awkward predicaments, well in advance of requirements, and reduced the cost from 2*s.* 6*d.* to 2½*d.* per acre.

He also undertook the office of Colonial Engineer, where he introduced important economies. His surveys embraced the whole of the known parts of South Australia and, in addition, a considerable part which he explored for the first time. Captain Frome returned on sick leave in 1849, and in 1850 was in command of the operations at Seaford, in which 24,000 lbs. of gunpowder, the largest charge hitherto exploded at one time, were employed to throw a portion of the cliff into the sea so as to form a groin or embankment against the creeping of the shingle. In 1851 he was appointed Surveyor-General of Mauritius, but resigned in 1858 on account of ill-health brought on as before by arduous duties in a tropical climate.

In 1858 he made, at the request of the Colonial Office, a full report on the civil and military engineering questions which had arisen on account of the recent inroads of the sea in the Island of Heligoland, and this report has been the basis of the subsequent works on the island. In 1859 he was appointed Commanding Royal Engineer in Ireland, and in 1863, on the resignation by Sir John Burgoyne of the post of Her Majesty's Inspector-General of Fortifications, Colonel Frome was named to succeed him; but on the Treasury raising some difficulties as to Sir John Burgoyne's retiring pension, Sir John was re-appointed to the post for a further five years, and Major-General Frome went out as Commanding Royal Engineer to Gibraltar, where as senior officer of the garrison he replaced the Governor, Sir Richard Airy, as Governor in his absence on leave at various times, and on the definite retirement of Sir John Burgoyne in 1868 he succeeded that officer as Her Majesty's Inspector-General of Fortifications and Head of the Corps of Royal Engineers. He continued as Inspector-General till the office was abolished by Mr. Cardwell, when he was offered and accepted the Governorship of Guernsey. Since his retirement from active service he has retained the keenest interest in his old profession.

General Frome was elected an Associate of the Institution on the 15th of May, 1838, and was unanimously nominated by the Council as an Honorary Member, to which position he was elected on the 4th of February, 1890. He lived to express, through his daughter, his gratification at the honour done him, but a few days later his long and distinguished career was peacefully ended.

WILSON BELL was born at Glasgow on the 29th of May, 1839. His father, Thomas Bell, was a merchant of that city, and the son was educated first at the Grammar School at Renfrew, and subsequently at the Grammar School at Paisley. He was articled to Mr. Martin, a civil engineer practising in Glasgow, and had the advantage of attending the engineering classes under Professor Rankine at Glasgow University.

After the expiration of his pupilage he obtained an appointment on the Recife and São Francisco Railway, in the Province of Pernambuco, Brazil, of which the late Mr. W. M. Peniston was the Chief Engineer. The coast section of the line had then been completed and opened for traffic, and Mr. Bell was posted to one of the upper sections as Assistant Engineer, remaining there till the undertaking was completed. On coming home in 1863, he accepted employment in Russia, but being offered an appointment on the staff of the Great Indian Peninsula Railway he returned to England, leaving for India in March, 1864. He was first employed as Assistant Engineer on survey work between Sholapur and Hyderabad, and later on the construction of the line between Sholapur and Gulburga under Mr. Sherwood, Messrs. Adamson and Clowser being the contractors. In May 1867 he was transferred to the section between Gulburga and Raichur, under Messrs. R. M. Brereton and J. Preston. In March 1868 he was promoted, and in February 1869 was appointed District Engineer of the portion of the railway between the Bhore Ghât and the junction with the Madras Railway at Raichur. His reputation as an able and energetic officer was now well established, and in order to secure the earliest possible opening for the line, he was placed by the late Mr. H. P. Le Mesurier, C.S.I., in independent charge of the unfinished construction works between Sholapur and Gulburga, in addition to the ordinary duties of his office. The confidence placed in him was fully justified by the result, the works were pushed on with vigour, and the value of his labours was fully recognized by the Board of Directors, the agent, and the Government of Bombay. In a resolution of Government, dated the 17th of January, 1870, it was stated:—

> "It has given Government pleasure to note the remarkable progress of the works between Sholapur and Gulburga during the last three months. The arrangements made by the agent, and the energy and vigour with which the works were carried out by the resident and assistant engineers, have been most creditable. If a proposal is made by the agent and London Board of the company to recognise by a pecuniary reward the exertions of the officers engaged on the contract, Government will be prepared to accede to a special grant of Rs. 10,000 being made for distribution between the Resident Engineer, Mr. Wilson Bell, and the different assistant engineers on the contract in a proportion to be determined by the agent."

When in April, 1874, the post of Chief Resident Engineer of the company in India became vacant, on the retirement of Mr. J. R. Manning, Mr. Bell was selected to fill it, and he continued to hold it till his death, having twice in the interval held the appointment of Acting Agent of the Company during the absence on furlough of Mr. G. A. Barnett, C.I.E. In 1881 the first signs of failing health made themselves apparent, and Mr. Bell was compelled to take leave on sick certificate for seventeen months. He returned to India with improved health, but in 1886 he was again obliged to leave the country, his many friends then feeling that he would never again be able to resume his duties. He did, however, return to India in November 1887, but in completely broken health, medical opinion being that the stimulus of work would probably lengthen his life. His indomitable spirit sustained him, and of no man can it be more truly said that he died in harness. His mental powers remained unclouded to the end, and he faced the inevitable with singular courage and resolution.

As an engineer Mr. Bell's reputation must rest on the results of his labours as a maintenance officer. It chanced that there are no great works the conception of which he can claim. The best years of his professional life were spent as Chief Engineer of the parent Indian railway, and from end to end it bears the impress of his well-matured judgment and earnest devotion to duty. In the system of continuous renewals, in the vastly improved class of work he insisted on, in the order and system introduced in the records of maintenance expenditure, and in the leading position held by the railway among Indian lines, are the proofs of his grasp of professional questions, and the breadth and soundness of his views. A man of an extremely self-reliant type, and himself an unsparing critic, he seldom sought the advice of others, and was careless of adverse opinion. He read little, was a deep thinker, and having formed his judgment on the question before him he carried out his views unflinchingly. A genial host, a staunch friend, prompt to recognise in others the gifts that were his own, he was followed to his grave by the regrets of a wide circle of friends. The Company he served so well lost in him a loyal and faithful servant, and an able adviser.

Mr. Bell was a Fellow of the University of Bombay, and was nominated by Government in 1883 as a member of the Municipal Corporation. He was much pressed by Government to accept the Chairmanship of the Town Council, but being always unwilling to step outside the immediate sphere of his duties, he declined the position. He was a member of the Commission appointed by the Government of Bombay in 1877 to determine the best scheme for the drainage of the city.

Mr. Bell died at Lanoli on the 27th of November, 1888, and was by his own desire interred in the churchyard at Igutpuri, the picturesque station at the summit of the Thul Ghât incline. He was elected a member of this Institution on the 3rd of February, 1874.

ALAN BREBNER, son of Alexander Brebner, builder, was born at Edinburgh, in August, 1826. He was educated at the High School of Edinburgh, and before beginning his studies as a civil engineer, served a regular apprenticeship as a mason, thereby gaining a knowledge of building construction, which served him in good stead in his professional life. He became a pupil of Messrs. D. and T. Stevenson, civil engineers, Edinburgh, and with only a short absence between 1848 and 1850, when he acted as resident engineer in carrying through a portion of the railway works between Thornton and Dunfermline, he was identified with the firm, first as resident engineer in the erection of harbour, lighthouse, and other works; then as assistant, and latterly until his death, as a member of the firm. He acted as resident engineer in the erection of the temporary lighthouses at Whalsay, and at North Unst in 1854, the latter being a difficult and arduous work. After this he was occupied with the works of the firm, namely, the design and execution of river improvements, harbours, docks, lighthouses, and other engineering works.

Mr. Brebner devoted special attention to lighthouse construction and optics, and devised several new arrangements of dioptric apparatus, and when making calculations in connection with them, invented a "refraction protractor,"[1] by means of which prisms for lighthouses can be laid down in about half the time formerly required to protract them. During Mr. Brebner's connection with the firm, schemes for lighting the whole of the coasts of Japan and New Zealand were matured, and are now being carried out. Mr. Brebner, who, at that time was Messrs. Stevenson's principal assistant, took a special charge in connection with the erection of the Rock Towers at the Chickens and Dhu Heartach—works of no ordinary difficulty.

Mr. Brebner took a lively interest in the experiments which led to the use of paraffin as an illuminant in lighthouses, which has so largely decreased the cost of maintenance, and increased the intensity of the lights, rendering it possible to employ burners of larger diameter and greater initial power. In engineering matters, Mr. Brebner's evidence in arbitrations or courts of law, and before Parliamentary committees, was always given after careful study and conscientious preparation, and he spared no pains to get at facts from which to draw sound conclusions.

He rarely wrote on matters connected with his profession, but gave valuable aid to the members of his firm in the preparation of their books on "Harbours," "Canal and River Engineering," and "Lighthouse Construction and Illumination" all of which have become standard works, and have passed through several editions. When he did write on professional subjects it was with full knowledge and experience, as in his Paper on "Modern Harbour Construction," read before the Royal Scottish Society of Arts, in July, 1887, and for which he was awarded a medal. One of the last pieces of business he was engaged in along with the members of the firm—Mr. David A. Stevenson and Mr. Charles A. Stevenson—was the project of connecting the East and West Coasts of Scotland by a ship canal capable of passing the largest merchantmen and Her Majesty's ships of war.

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[1] Minutes of Proceedings Inst. C.E., vol. xxviii. p. 34.

Mr. Brebner was elected a member of the Institution in 1878; he was a man of sound judgment, and had the rare faculty of rapidly and accurately grasping the details of engineering problems. He had a ready and retentive memory, and was expert in methods of calculation. Of a quiet and unobtrusive disposition, he rarely interfered in matters outside the profession. Though reserved in manner, he was a warm-hearted friend, able and willing to give sound advice. He was in his usual health till about three weeks before his death, when he had a sudden attack of hemorrhage, and though this was overcome, his strength failed, and he died suddenly on the 5th March last.

JOHN AUGUSTUS ARNOLD BUCHHOLZ was born on October 12th, 1846, at Potsdam, in Prussia. He was the son of Gustav Adolph Buchholz, an engineer of great ability, the printing press, sold by Baron Bunsen to *The Times*, by which that paper was for many years printed, being one of his many inventions. It is difficult to describe the part played by Arnold Buchholz in revolutionizing the old system of flour-milling, and introducing the new one, without first briefly referring to his father, Gustav Adolph Buchholz and his work. He was one of the first to introduce the roller-mill and semolina milling into England, and was the inventor of a considerable number of machines for decorticating or hulling and grinding flour. The process of the gradual reduction of the grain to flour, now almost universal, met with little success when introduced twenty-five years ago, the prejudices of the old school of millers, and the immense amount of capital invested in the old process, being too much for two men to overcome.[1] Gustav Adolph Buchholz died at Leipzig, in September, 1888, and a very fair, though far from complete account of him, appeared shortly after his death in *The Miller*.[2]

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[1] A description of the roller-mill system will be found in the Minutes of Proceedings Inst. C.E., vol. lxx. pp. 162–270, and vol. xc. p. 366.

[2] *The Miller*, vol. xiv. p. 502 (Feb. 4, 1889).

Arnold Buchholz received his early education in Germany and England. He became especially proficient in mathematics, and wrote and spoke with fluency in the English, French, and German languages. After leaving school he was articled from 1863 till 1868 to his father, during which time he was engaged upon the design and construction of machinery for the following mills:—Messrs. Russell and Sons, Limerick; S. Sealy Allin, Middleton, Cork; and G. A. Buchholz's mills, Senlis, near Paris, in which Mr. Allin was likewise interested. He then left his father, and after some months of hardship, obtained a post in an engineer's office, in Paris, where he assisted in the designs of some of the earlier mitrailleuses. After a few months in this office he obtained the appointment of assistant, and subsequently, of principal engineer to the Wire-Tramways Company, and was engaged in the construction of tramways in France and in Austria. From 1871 to 1873, Arnold Buchholz was associated with his father in designing machinery for some large mills in Hanover. From 1873 to 1877 he was engaged in experimenting upon, and in patenting improvements in milling machinery, and during a portion of this time he also edited *The Corn Trade Journal*, where many papers from his pen testify to his scientific attainments, and to his power of analyzing difficult mathematical processes. For complicated mills he elaborated a concise diagrammatic system of notation, which, while relieving the memory of the endless details of a mill, much simplified the process of design by showing at a glance what was happening to a product at any point in a mill, pocketbooks, specially ruled, being used for noting down the details of the process in mills which were to be altered or redesigned. From 1876 to 1882 he was engaged in designing and building flour-mills for Messrs. Barlow and Sons, Bilston; Messrs. Fairclough and Sons, Warrington; Messrs. John Ure and Sons, Glasgow; Messrs. Frost and Sons, Chester; Messrs. Reynolds and Hasselgrove, Wakefield. From 1882 on to 1888 he was engaged in the design of several large mills, among others, one for Messrs. William Baker and Sons, of Bristol.

The difficulty with roller-mills has been to keep the rollers apart when the feed stops, contact being a thing to be feared, and for this purpose, devices have been applied to throw the rollers apart when the feed gets thin. Mr. Buchholz applied himself to designing an adjustment for axles and bearings, which, while securing the rollers against possible contact in the event of the feed ceasing or becoming thin, should at the same time admit of adjustment to $\frac{1}{10,000}$ inch, for experiments had shown him that a very minute variation in the distance of the roller apart makes a great difference in the quality and quantity of the semolina and fine flour produced in the "cracking" of brans to which fine stuff is adhering. Whilst fulfilling these two conditions, the arrangement was to allow of the rollers moving apart temporarily whenever any abnormally large foreign object was passing through with the feed. An adjustment which would fulfil these conditions was patented in 1883.

In 1884, Mr. Arnold Buchholz was appointed Consulting Engineer for flour milling to Messrs. Greenwood and Batley, and he continued in this post up to the time of his death, while practising also on his own account as a Consulting Engineer. His great experience as an inventor, and as one who had seen both the professional and commercial sides of milling engineering, caused his advice to be much sought after in cases of disputed patents, and his evidence as an expert in these matters always carried weight in the witness-box.

Early in 1889 he commenced practice in London and Bristol, residing in the latter place, where he acted as consulting engineer to Messrs. William Baker and Sons. Up to the last he continued inventing, patenting, and experimenting, his efforts being particularly directed to the production of a machine for crushing semolina and making flour. The object he had in view was to supersede roller mills for crushing semolinas, the efficiency of these machines being much lowered by the entrance of air-currents with the feed. He therefore planned a disc mill in which the principle of his adjustment for parallel axes was applied to concentric axes.

Mr. Buchholz did not live to perfect this machine, and experiments brought to light certain difficulties which, however, he felt confident of overcoming. He invented several mills, machines for pulling grains, sorting machines and many other milling machines, as is evidenced by the number of patents taken out in his lifetime.

An engineer of such experience as Mr. Buchholz could not fail to recognize the great difficulty and expense of converting mills on the old plan to the new system, and also the costliness of mills built according to the requirements of the modern system; he accordingly devoted much time and thought and money to designing a system which should combine the simplicity of the old method with the good results of the new, but after experiments extending over three years he gave up the attempt, convinced that gradual reduction or semolina milling, which he had helped to perfect, was correct and likely to be permanently adopted.

Mr. Arnold Buchholz was a naturalized Englishman, and many years of residence in this country had given him the tastes of one. In character he was upright, warm-hearted, and generous. one who could forgive an injury frankly, and, what is rarer, forget it. Proud and quick-tempered, jealous for his rights where he thought them encroached upon, knowing no compromise for the sake of expediency or prospective gain, he was not the man to smooth the difficulties that appeared in the hard uphill path of early manhood; and these idiosyncracies of character, of which he was well aware, caused him to be much misrepresented and misunderstood. Circumstances prevented the father and son from acting together for their mutual benefit at the time when the former's patents had to be relinquished, and Arnold Buchholz saw others reap the fruits of his father's patents and their united labours. Bright hopes alternating with keen disappointments and the loss of his father in 1888, left their mark upon a temperament alternately too sanguine and too despondent. Several bad falls in the hunting-field, of which he was passionately fond, and where he rode straight and daringly, undoubtedly contributed to, if they were not the actual cause of, his untimely death by his own hand at Clifton on the 13th of May, 1890.

He was elected a member of the Institution on the 7th of May, 1889.

JOHN CARRICK[1] was born at Denny, Stirlingshire, on the 6th of May, 1819. After completing his education, he served an apprenticeship with the late Mr. John Bryce, architect, was then assistant to the late Mr. Herbertson, and also spent a short time in England and in foreign travel acquiring further experience in the profession. Returning to Glasgow, he began business in partnership with a Mr. Brown, but the partnership lasted only a few years. Soon afterwards Mr. Carrick entered the service of the Glasgow Corporation, succeeding the late Mr. Hume in the office then known as superintendent of streets. This was in the beginning of the year 1844; thereafter until his death Mr. Carrick served the corporation as city architect. Mr. James Lumsden was then Lord Provost; and the amalgamation of the burghs of Calton, Gorbals, and Anderston, which was carried through under his auspices, had just been accomplished. It is a somewhat singular coincidence that the first appearance of Mr. Carrick before a Parliamentary Committee was as a witness for the city in connection with the amalgamation bill, and that the last work on which he was engaged was in preparing evidence in favour of the annexation scheme of the present year.

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[1] This memoir is abridged from the *Glasgow Herald*, May 3, 1890.

Very early in his career, Mr. Carrick came to be known as a man of sound judgment, with wide and far-reaching views on all subjects of civic economy; and when the City Improvement Scheme was projected he was one of its most cordial supporters. The credit of the inception of this scheme was due to the late ex-Lord Provost Blackie, the late Sir James Watson, the late Mr. Campbell of Tullichewan, Sir Archibald Orr Ewing, and other well-known philanthropists; but Mr. Carrick lent valuable assistance in maturing and seeing it carried through Parliament. The duty of selecting the areas to be dealt with, and of determining the method in which they were to be operated upon, were largely his care. Into this work he entered with his whole heart, bestowing upon it an unlimited amount of time and attention. In the course of a few years, as the result of the action of the trustees, and through the effect of the railway operations then in progress, he had the satisfaction of seeing that large portion of the central district, which had been looked upon as a danger and disgrace to Glasgow, in character and appearance completely transformed. Not yet, however, has full effect been given to Mr. Carrick's views for the improvement of this part of the city; and one of the last pieces of work on which he was employed was the preparation of the elevation for the houses on the west side of the new Place to be formed at the Cross in connection with the construction of the Central Railway. Other improvements executed or projected about this time were the formation of the new squares at Bridgeton Cross and Gorbals Cross; the opening up of a fine approach to the cathedral from the south by way of John Knox Street; the covering in of the Molendinar Burn, and the formation of a street along its course; and the construction of the Alexandra Parade. Mention ought also to be made in this connection of the laying out of the lands of Oatlands and Overnewton for the erection of workmen's houses, which was also the work of Mr. Carrick. While utilizing the ground to full advantage, he took care to see that the streets were wide and airy, providing in addition a large square in the centre of the property—an arrangement which it is to be regretted has not been more largely adopted by urban proprietors. Mr. Carrick was also the architect of the workmen's dwellings in the Saltmarket, and of the model lodging-houses erected by the Improvement Trustees. The design of the lodging-houses was quite original in character.

They have been found to answer fully the purpose for which they were constructed, namely—to provide a comfortable home for the loose floating population found in all large cities; and they have been highly commended by many distinguished visitors engaged in similar philanthropic works. The success of the experiment in Saltmarket was not less signal and complete. To Mr. Carrick, too, fell the duty of preparing the designs for the fever and small-pox hospitals at Belvidere. These are constructed on the pavilion principle, and in the matter of arrangement are admitted to be among the most perfect institutions of the kind in the kingdom. The Western Police Office in Cranstonhill, the Marine Police Office in M'Alpine Street, the Eastern Police Office in Tobago Street, and the Northern Police Office at present in course of erection in Maitland Street, testify to his architectural taste and skill. A few years ago, when the attention of the magistrates of the city was directed to the necessity of improving the accesses of the public halls of Glasgow, Mr. Carrick entirely remodelled the City Hall. The Bazaar was also reconstructed at the same time, and what previously had been a very inconvenient corner at Bell Street and Candleriggs was transformed into a spacious thoroughfare.

The question of the disposal of sewage necessarily received a large share of Mr. Carrick's attention, and with the immense network of sewers all over the city he had the most familiar acquaintance. Indeed, the entire existing sewerage system, some portions of it involving operations of great magnitude, may be said to have been laid out according to his plans and under his personal supervision. The great intercepting sewer in course of construction by the Central Railway Company, and which may yet form part of a scheme for the purification of the Clyde, is from a design submitted for his approval. The tramway routes were also laid out under his direction.

Sir Joseph Paxton prepared the design of the Kelvingrove Park, acquired in 1852, and the Queen's Park, but the working out of the details was largely due to Mr. Carrick. He also drew the feuing plan of the ground adjacent to the Queen's Park and the Alexandra Park. Among Mr. Carrick's later works was the preparation of the block plan of the new City Chambers in George Square. Mr. Carrick submitted more than one scheme to successive town councils for a museum and picture-galleries worthy of the city. The most recent was that now under consideration of the Parks and Galleries Committee and the Committee of the Exhibition Executive. It contemplates the erection of suitable buildings on the site of the International Exhibition in Kelvingrove Park.

Mr. Carrick was held in the highest respect by his professional brethren; and among municipal authorities throughout the United Kingdom he had an established reputation. From all parts of the country deputations on frequent occasions inspected the works which he had been instrumental in creating; and he was often applied to for counsel and advice. He was elected a Member of the Institution on the 10th of April, 1877, and was also a member of the Glasgow Institute of Architects, and a Fellow of the Royal Institute of British Architects. He died on the 2nd of May, 1890.

THOMAS RIDLEY HETHERINGTON was born in Glasgow on the 5th of April, 1835, but the family removed to Manchester in the following year. The father, John Hetherington, was for some time associated with the late Sir William Fairbairn in marine work on the Thames, and was the inventor of the two-flue, commonly called the Lancashire, boiler, for which he and Sir William had a joint patent. In 1836 Mr. Hetherington, senior, founded the business of John Hetherington and Sons, in which the subject of this notice was a partner for thirty-four years, having previously served his time in its shops and drawing-office. This firm, in process of time, took rank among the most extensive producers of machine-tools in the kingdom. They had large dealings with Her Majesty's dockyards and arsenal, for which they designed and constructed special tools. They were also concerned with most of the large railways in this country and in India, particularly for heavy tools, and machinery for the transmission of power; also in power-driven cranes. Along with the tool and engineering business was associated another establishment, employing one thousand men, where cotton-spinning machinery was made, and where plans for large factories, to be engaged in this industry, were designed, and subsequently carried out. In both of these departments Mr. T. R. Hetherington designed and perfected many special tools and appliances.

Although his modest and retiring disposition indisposed him to take part in the public life of Manchester, he was yet one of the best-known and most respected of its citizens, and his death, on the 10th of December, 1889, when still in the prime of life, caused general regret.

Mr. Hetherington was elected a Member on the 2nd of March, 1886.

WILLIAM INGLIS was born on the 10th of May, 1835, in Ottawa, Canada, of Scotch parentage, his father having settled in Canada about the year 1825.

In the year 1852 young Inglis was apprenticed to Mr. Gilbert, of the St. Lawrence Engine Works, Montreal, with whom he remained until 1856, when he came to this country and entered the works of Messrs. R. Napier and Sons, Glasgow. Here he remained for two years, and during that time, he attended the Engineering Classes of Professor Rankine, at the Glasgow University. On leaving Glasgow, in 1858, Mr. Inglis entered the locomotive works of Messrs. Robert Stephenson and Co., of Newcastle-on-Tyne, and in 1860 he returned to Canada, where he was engaged for two years in designing and erecting machinery, including a walking-beam paddle-engine, fitted with Corliss valves, for the river paddle-steamer "Montreal."

Mr. Inglis also superintended the building of the hull of the "Montreal," the first iron passenger steamer built in Canada, and which is yet running on the St. Lawrence.

In 1863 Mr. Inglis came back to England, opening an office in Edinburgh as a Consulting Engineer, and during his residence there he patented and erected an inclined water-tube boiler, which is still in good working condition. In 1864 Mr. Inglis removed his offices to Manchester, and designed an improved type of Corliss engine, with which his name is so closely and deservedly associated.

The large development of the Corliss valve and gear during the past twenty years is mainly due to the "Montreal" and to Mr. Inglis, and it may be worth recording the circumstances that support this statement. In 1861 Mr. J. F. Spencer, during a visit to Canada, met Mr. Inglis in the engine-room of the "Montreal," and was impressed with the efficiency of the Corliss valves and gear, and on his return to England patented what is known as the "double-clip gear." In 1862–64 two pairs of horizontal Corliss mill-engines, of 400 HP. each, and two high-speed Corliss vertical engines, of 100 HP. each, were started by Mr. Spencer at Bradford and Blackburn, fitted with the improved gear. This type of engine excited great controversy for many years, and its present success, and almost universal adoption for large mill-engines, is greatly due to the energy and pertinacity with which Mr. Inglis upheld its advantages.

During his residence in Manchester Mr. Inglis superintended the construction (on the Clyde), and shipment in plates to Canada, of several large steamers. In December, 1867, he was appointed the engineering manager of, and ultimately a partner in, the Soho Iron-Works, Bolton, Messrs. Hick, Hargreaves and Co. being the first firm to manufacture the Corliss engine in this country under the Inglis and Spencer patents, and during the twenty-two years of Mr. Inglis's management he perfected many improvements in Corliss engines.

Nearly nine hundred Corliss engines were constructed at Soho for mills, &c., ranging in power from 50 to 10,000 HP., the latter being now in hand for the London Electric Supply Corporation, to indicate 5,000 HP. on each crank. This power (as developed on one crank) is unprecedented on land or sea.

Perhaps more than any one individual, Mr. Inglis was instrumental in bringing mill-engines to their present high state of perfection. He was an earnest advocate of high piston-speed, and high steam-pressure, and in fact of all the features which mark the most advanced practice in steam engineering. He was equally capable in designing other classes of machinery. His name is also well known in connection with cold-air machinery for the imported fresh-meat trade; and automatic Barring-engines for starting large engines, &c.

Personally Mr. Inglis was quiet and self-possessed in manner, kind and considerate, especially to those over whom he had authority. As a technical witness he was invaluable, many a case has been won largely owing to his evidence. In short, his death in the prime of his career, is a great loss to the profession.

Mr. Inglis died on April 22nd, 1890, after only a few days illness, of pneumonia, at his residence, Wilton Grange, Bolton. He was elected a Member of the Institution on the 5th of December, 1876.

FREDERICK JOPLING, born on the 11th of November 1855, was the third son of the late Thomas T. Jopling; after being educated at the Collegiate School, London, he entered the office of Messrs. Meik and Nesbitt, of Sunderland, as a pupil, serving a five years' term until 1876, during which time he was employed in preparing plans and surveying for many important works. Among these were the Hylton and Monkwearmouth Railway (on which work he was engaged until its completion), and a scheme for supplying the district of Bedlington with fresh water, which was subsequently carried out most successfully, Mr. Jopling acting as Resident Engineer. On the expiry of his pupilage, he entered the service of Mr. James Young, contractor, and was engaged for a year as contractor's engineer on the Bedlington Water-works. On the completion of these works he returned to Thomas Meik and Sons, which firm had succeeded that of Meik and Nesbitt. In their office for the next eighteen months he was engaged in the designs of various harbours in Scotland, including Burntisland, Ayr, and Bo'ness Docks, and in November 1878, he was appointed Resident Engineer on the latter work.

While at Bo'ness he competed, and gained first premium offered by the local authorities for the best scheme for a system of drainage for that town. In 1882 the whole of the harbour works were successfully finished and opened. In March of the same year Mr. Jopling received the appointment of Assistant Engineer under Mr. John Fowler, to the Tees Conservancy Commission, and in the following August entered upon his duties. Here a large sphere of varied work lay before him in the multitudinous works carried on by the Commission, notably the reclamation works, the South Gare Breakwater, and the dredging. On the 22nd of February, 1888, he was appointed to the sole charge of all the works of the Commission, Mr. Fowler being retained as consulting engineer, and on the death of that gentleman in October of the same year, he was appointed Engineer-in-chief. In this capacity he had charge of the many and varied works carried out by the Commissioners, including reclamation-embankments, the construction of the head of the south pier, design and construction of the north pier, erecting lighthouses, patent slipway, compressed-gas lighting-apparatus and beacons, alteration and extension of graving dock, widening and wharfing of the river at Stockton, and the removal of the old stone bridge and foundations, removal by blasting of obstructions in the upper reaches of the Tees, and the construction of a water-pipe bridge across the river, the laying of over 2½ miles of water-pipes, construction of a portion of the Tees Sideroad and railway, deep water berths, laying down additional moorings, and the whole of the dredging, employing a fleet of five dredgers, eight tugs and thirty-two hopper-barges, and the general maintenance of the river.

Unfortunately Mr. Jopling was not destined to live long enough to set his mark on these undertakings, as he died on the 17th of February, 1890, within eighteen months of his appointment. He was elected an Associate of the Institution on the 1st of March, 1881, and was transferred to Member on the 19th of March, 1889.

ALEXANDER MACKINTOSH, second son of the late William Mackintosh, of Geddes, Nairnshire (a Surgeon-Major in the Madras Army), was born in Edinburgh on the 8th July, 1820, and educated first at the Academy, and afterwards at the High School in that city. He was a pupil of the late Mr. Joseph Cubitt, V.P. Inst. C.E., and was engaged on the surveys of the South Eastern Railway, and afterwards upon the construction of that line, acting as Resident Engineer on one or more of the sections. In 1844 he was employed under the late Mr. Henry Robertson upon the surveys for the North Wales Minerals Railway from Saltney to Westminster Colliery at Ruabon, as well as several mineral branches to Minera, the Wheatsheaf, &c. This line formed the original portion of the Shrewsbury and Chester Railway which was opened in 1846 from Chester to Ruabon, and afterwards extended to Shrewsbury. Mr. Mackintosh was also engaged upon the construction of the line, the principal works being the well-known Dee viaduct over the vale of Llangollen, and the viaduct at Chirk. He was likewise engaged upon the Oswestry branch, and upon nearly all of the mineral lines which were made in the mining districts adjoining the Shrewsbury and Chester Railway. After the opening of the latter, he acted as its Resident Engineer, during which period he resided at Gresford. On the absorption of the Shrewsbury and Chester Railway by the Great Western Company in 1854, Mr. Mackintosh retained charge of the Chester lines, and also became Engineer of the joint stations of Shrewsbury and Chester under the Great Western Company, and was engineer for several extensions, new works, &c., for them in that district. In 1860 he was appointed principal engineer for the northern division of the Great Western Railway, taking up his headquarters at Reading, subsequently acting as principal Resident Engineer under the late Mr. Michael Lane, and at various times residing at Slough, Reading, and Worcester. In 1866 he resigned his position on the Great Western Railway in order to enter into partnership with his friend Mr. Henry Robertson, who was then M.P. for Shrewsbury. Messrs. Robertson and Macintosh practised as engineers at 5, Westminster Chambers, until the death of Mr. Henry Robertson. During the partnership they constructed the Vale of Llangollen Railway, the Bala and Dolgelly, the Bala and Festiniog, and other branches in the North Wales district, and also the important Central Wales Railway extension from Knighton in Radnorshire to Llandovery. This latter was a heavy line to construct, the principal works being Llanrwtyd viaduct, the Knighton viaduct, several tunnels and heavy earthworks. This line was subsequently acquired by the London and North Western Railway Company, and now forms that company's principal communication between South Wales, Manchester, and the North.

Mr. Mackintosh was an original shareholder, taking a leading part in the Minera Lead Company and the Minera Lime Company, of which he became a director. He was principally instrumental in forming the Wrexham Colliery Company, now one of the largest and most important collieries in North Wales; and of this company he was for many years chairman, and was a director at the time of his death. Mr. Mackintosh was a man of culture and of considerable attainments. He was very painstaking, and extremely methodical in all business affairs; quiet and unpretentious in his mode of living, and a man of sterling honour, thoroughly trusted by all his friends. Socially, he was an excellent companion, and possessed a rare fund of quaint Scotch humour. His health during the past ten years had been failing, and he died at Blackheath on the 23rd of March, 1890.

Mr. Mackintosh was elected a Graduate of the Institution on the 18th of February, 1840, and was transferred to full Member on the 13th of February, 1849.

ALFRED PENNY, who came of a Somersetshire family, was born in London on the 11th of October, 1811. Not much is known of his early career, but it is believed that his engineering experience dates from 1844, when he became a partner in an establishment for the manufacture of machinery for paper-making, steam-engines, &c. In this venture Mr. Penny was not successful, and in 1849 he was agent for Messrs. James Oakes & Company, ironmasters of Alfreton, Derbyshire, in which capacity he was brought into contact with numerous provincial gas companies, and persons interested in gas. In 1850 he began business on his own account as a gas engineer, and for a time also as contractor, having established the Wenlock Ironworks, in the Wharf Road, City Road. Here he constructed gas-holders and general gas-plant for gas-works in different parts of the country. In these enterprises he was associated with all the pioneers of the gas industry, whose names are now almost forgotten. From building small works, Mr. Penny became lessee of undertakings of his own creation, and also of larger ones—including the works at Whitehaven, Portsmouth and Oporto. In this way Mr. Penny made a reputation as an expert in gas management, which led to appointments in a consultative capacity for many gas companies and corporations in Parliament, and for general purposes. He established his reputation in the committee rooms in connection with the Dundee Bill; and from 1865 onward, for twenty years, he was a leader in this branch of the profession, and was always engaged in important cases. His integrity was universally recognized; and his kindliness and willingness to help those who came to him for advice and assistance were conspicuous. He was a hard and conscientious worker, never failing to satisfy his numerous clients that the best had been done for them, even when their case happened to be unsuccessful. Of late years his tall slightly-bent figure was less frequently to be seen in the corridors of the Houses of Parliament; but he did not relinquish the practice of his profession until October 1887, when he had attained the age of seventy-six. Until then he had enjoyed almost uninterrupted good health, but at that time he contracted a severe illness from which he never quite recovered, dying on the 4th of March, 1890.

Mr. Penny was never a self-assertive man, and his work being done he dropped out of the ranks quietly. He was elected an Associate of the Institution on the 24th of May in 1859, and was transferred to full Membership on the 30th of October, 1877.

JOHN STANDFIELD was born at King's Newnham, Warwickshire, on the 23rd of July, 1838. His early professional experience was gained in Canada, where he was employed on the construction of the Grand Trunk Railway, chiefly in connection with the Victoria Bridge, Montreal. On returning to England in 1861, he had the superintendence of sinking the caissons for the foundations of the London, Chatham and Dover Railway Company's Blackfriars Bridge; and in 1864–66, Messrs. Peto and Betts entrusted him with the same duties on the widening of the Victoria Railway Bridge at Pimlico. Mr. Standfield subsequently had charge of Mr. Rammel's Pneumatic Railway under the Thames from Waterloo to Whitehall, but on the abandonment of the works he devoted his time to the study and development of various inventions, which he afterwards turned to account, until in April 1869 he proceeded to Bombay, as Resident Engineer for Mr. Edwin Clark's hydraulic-lift graving-dock at Hog Island, which work he successfully carried out, returning to England in 1872. Shortly after this, Mr. Standfield entered into partnership with Mr. Latimer Clark, and devoted himself more particularly to the study and improvement of floating-docks and hydraulic canal-lifts, and, in conjunction with Mr. Clark, became the inventor and patentee of several novelties in connection with this branch of engineering. The most important of these inventions were: the gridiron depositing-docks, constructed for the Russian Government, at Nicolaieff, on the Black Sea, and at Vladivostok, the designs for which Mr. Standfield submitted and explained to the late Emperor of Russia; a gridiron depositing-dock at Barrow-in-Furness; hydraulic canal-lifts erected at Fontinettes, near St. Omer, in France, and at La Louvière, in Belgium; and the off-shore dock, the first of which was built at Messrs. Standfield and Clark's engineering works, at Grays, Essex, and was launched and towed to Cardiff in June 1887; a second dock of the same class was afterwards laid down, from Mr. Standfield's designs, at Hamburg, and is believed to be the most rapidly worked floating-dock in existence.

Mr. Standfield also gave much attention to the raising and recovery of sunken vessels, and exhibited working models of his patent appliances to H.R.H. the Prince of Wales at the Westminster Aquarium. He was applied to by the owners of the s.s. "Austral," of the Orient Line, a vessel of 5,588 tons register, when she sank while coaling in Sydney Harbour in 1882, and succeeded in raising her without injury. Mr. Standfield contributed a Paper[1] on this work to the Institution, for which he was awarded a Telford premium.

It was, unfortunately, in connection with this class of operations that his useful and energetic career was brought to a premature close, at the age of fifty-one.

In 1888 the s.s. "Ville de Calais," a petroleum tank vessel, exploded and sank in Calais Harbour, and Mr. Standfield some time afterwards purchased the wreck, with the view of raising the undamaged portion which contained the engines and boilers. This, after a considerable expenditure of time and money, he succeeded in doing, after cutting off the damaged bow-portion of the steamer and building a bulkhead across her. On Saturday evening, March 1st, 1890, she was got under steam, and also taken in tow by a powerful tug, with the intention of bringing her to England. Early on Sunday morning, 2nd March, a sudden storm arose, and, breaking away from the tug, the vessel became unmanageable; an attempt was made to leave her in a boat, which capsized in lowering, and Mr. Standfield and three of the crew lost their lives. The vessel shortly afterwards foundered about a mile off Margate. Mr. Standfield's body was recovered the following morning off Deal, and was interred at Norwood cemetery on the 8th of March.

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[1] Minutes of Proceedings Inst. C.E., vol. lxxiv. p. 246.

Mr. Standfield was elected a Member of the Institution on the 12th of April, 1881.

HENRY PALFREY STEPHENSON was born on the 27th of March, 1826, at Portobello, near Edinburgh, where his father, Major Stephenson, of the 6th Dragoon Guards, was stationed. The first years of his life were passed in Ireland, but at six years of age he was taken to Cheltenham, and remained there for two or three years. He subsequently resided up to the age of fourteen at his father's estate, Boscombe Manor, near Bournemouth. He was educated at a private school at Twickenham, and afterwards, in 1842, in order to prepare himself for the profession, he entered as a student at the College for Civil Engineers, Putney. He remained there for five years, and among his fellow-students were: Sir Guilford Molesworth, Mr. W. J. Kingsbury, Mr. E. Riley, Mr. E. Pontifex, Mr. M. Woodifield, and Mr. Robert King. The Principal of the College then was the Rev. Morgan Cowie, D.D., Dean of Exeter, and among the Professors were Sir Lyon Playfair and Professor Codrington. Mr. Stephenson acquitted himself so well at this College that he was one of the few students upon whom a Diploma would have been conferred had the College been continued and received a charter; in consequence, however, of its ceasing to exist, Mr. Stephenson was induced to establish what was at first called the Putney Club, but which was afterwards converted into the Society of Engineers, his idea having been to establish a Society for the encouragement of young students in the profession, and thus enable them to take a more active part in the proceedings of the senior Society, which did not, at that time, accept students.

While at College, he was engaged for a short time in the survey of projected lines of railway in Ireland, then creating such a stir in financial and engineering circles. After leaving the Putney College, he was, in 1849, employed on the extension of the Sunderland docks, under the late Mr. John Murray. In 1850 he entered into partnership with the late Mr. William Dredge, and the firm of Dredge and Stephenson was for some time engaged in the construction of bridges in England and Wales. The partnership, however, was not of long duration, and in 1852 Mr. Stephenson devoted himself to private practice.

From the foregoing, it will be seen that Mr. Stephenson's training essentially fitted him for a dock and railway engineer, and it was in this capacity that he commenced private practice. Among other works which occupied his attention about this time was the designing—in conjunction with his old fellow-collegian, Mr. Robert King—of several iron bridges which were erected in Singapore, Adelaide, the Argentine Republic, and Natal, under the title of the Iron-Bridge Association. Altogether he erected about twenty bridges, ten of them being in England or Scotland. Conjointly they were also engaged in the arbitration case in which Dr. Oakes, of Cherry Hinton, Cambridge, claimed compensation for the proposed abstraction of water from springs on his property by the Cambridge Water Company; and also in the arbitration case with respect to the sale of the West London Junction Gas Company, Wormwood Scrubbs, to the Great Western Railway Company. He was Engineer to the Ware, Hadham, and Buntingford Railway Company, the works for which he designed and carried out, and which was afterwards sold to the Great Eastern (then the Eastern Counties) Railway Company. He also designed and carried out sewerage and drainage works at Narborough, Norfolk, so as to utilize the sewage on the estate.

In 1858 Mr. Stephenson, with that intuitive insight which he showed throughout his professional life, saw that the lighting of towns by coal-gas would be greatly promoted, and therefore gave much attention to this subject. Consequently he designed and carried out successfully, with the help of his then pupil and assistant, Mr. Charles Gandon, gas-works in the following towns in Germany, viz., Tilsit, Naumburg, Ludwigsburg, and Zeitz; and he was also consulted as to numerous gas-undertakings in Holland, and, as Director and Consulting Engineer, gave valuable advice to a great number of gas companies; so that perhaps no man now living can lay claim to having served in the former capacity to the same extent as Mr. Stephenson.

Previous to his retirement about 1882, on account of ill health, he was a Director of the Bahia, Bombay, Singapore, Georgetown, Cagliari and Para Gas Companies, and until the day of his death he was on the Boards of the Crystal Palace District and Croydon Gas Companies. He was connected with the Singapore and with the Georgetown (British Guiana) Gas Companies from their origination, and was Chairman of both. With regard to the Crystal Palace District Gas Company, he was associated with the late Sir

Erasmus Wilson and other gentlemen in its promotion (the first general meeting being held in March, 1858), and for many years he was Chairman, and by his exertions it was established on a firm basis. Mr. Stephenson served on the board of the Croydon Commercial Gas and Coke Company from 1879. As recently as February 19, 1890, he retired from the office by rotation; and although it was announced that he could seldom attend the Board meetings, the shareholders showed the measure of confidence they reposed in him by re-electing him without a single dissentient. The deep interest which he took in the concern, up to the day of his death, is evidenced by the fact that, whenever it was possible for him to be wheeled out in his bath-chair, he invariably desired to be first taken round the works of this Company, which are situated only a short distance from his house.

He was elected an Associate of the Institution on the 3rd of May, 1853, and was raised to Membership on the 23rd of February, 1864.

As a noteworthy incident in his life, it may be stated that, by his active and acute investigations some years ago into the affairs of the New Sombrero Phosphate Company, the shareholders, by the decision of the Law Courts, obtained their rights and recovered their money, and they consequently presented Mr. Stephenson with a handsome testimonial as a mark of their gratitude. The work in which Mr. Stephenson took most interest was the preparation and giving of evidence in arbitration cases, and in inquiries before Parliamentary Committees; and it was his intention, had his health been maintained, to devote himself almost exclusively to that class of practice.

Mr. Stephenson's personal characteristics were an unfailing courtesy and an extreme willingness to help his fellow-workers when appealed to for advice and assistance. Many men owe their successful careers to his kindly actions and thoughtful consideration. He did not forget those who had worked well for him or with him, and watched their progress with sincere gratification. Although he had been in failing health for several years, he never shirked work, and occupied himself up to the last with mastering all the facts of any business matter in which he was interested. His intellect was vigorous, and his brain singularly active, while his retentive memory enabled him to supply all the details of recent and long-past events.

Mr. Stephenson died on the 30th of April, 1890.

JAMES STUART SWALLOW was born on the 14th of December, 1849. He commenced his professional career as a pupil of Mr. T. Raper, Borough Engineer to the Scarborough Municipality, to whom he was articled for a term of four years from the 2nd of January, 1865, and at the expiration of two years of his pupilage, he was engaged as Assistant Borough Engineer. This position he held for five years, receiving at the end of that period the most flattering testimonials from his Chief, as well as the Members of the Town Council, with whom he came daily into contact. He resigned the post to become Chief Assistant to Mr. G. Wells Owen, Engineer to the Severn Bridge and Railways, and sundry lines in South Wales. In 1873, Mr. (now Sir) Charles Hutton Gregory, Past President Inst. C.E., offered Mr. Swallow the appointment of Assistant Engineer on the Cape Government Railways, which was accepted. In this service he remained until December, 1880, being promoted to the rank of Acting District Engineer in August, 1875, and in September, 1876, he was further advanced to the position of District Engineer. On the completion of the work his employment at the Cape terminated; but an extension of railways having been authorized, an opportunity arose for a renewal of his services, and he re-entered the department in June, 1882, as District Engineer for Railway Surveys and Construction on the Midland system, being afterwards employed to survey the proposed lines between Morsel Bay and Oadshoom. On the 1st of July of the year named, he was appointed to take charge of the construction of a length of 30 miles of line on the western system. In June, 1884, Mr. Swallow returned to England on urgent family affairs. During his career at the Cape his zeal and industry in the discharge of his professional duties were most highly appreciated by his superiors, and his tact and urbanity rendered him beloved by all those who were associated with him.

After a short stay in England, he went to India as Engineer on the Southern Mahratta Railway, returning thence on the completion of the works on which he was engaged, in the autumn of 1888.

Shortly after his return home from this post he went to Uruguay in the employment of Messrs. Perry, Cutbill, De Lungo and Co. the contractors for railways in that republic. On the 6th of April, of this 1890, after a short illness of ten days, he died at Rocha, Uruguay, and thus terminated, at the early age of forty, a career full of brilliant promise.

Mr. Swallow was elected an Associate of the Institution on the 5th of March, 1878, and was transferred to Member on the 4th of March, 1884.

RALPH ARCHBOULD was born at Islip, in Northamptonshire, on the 25th of September, 1856. In 1874 he was articled as a pupil to Mr. William Adams, who was at that time Locomotive Superintendent of the Great Eastern Railway. Four years later he accompanied Mr. Adams to the London and South Western Railway, where he was employed in connection with the many extensive alterations which that gentleman made at the Nine Elms Works in the laying down and equipment of the new workshops. After being here a few years he went to America in order to visit various works in that country, and to get an insight into American shop practice. On his return he entered the firm of Messrs. Woodhouse and Rawson, Electricians, as works manager, and held that post for several years until bad health compelled him to retire. He died on the 6th of December, 1889, at the early age of thirty-three.

Mr. Archbould was elected an Associate Member on the 6th of December, 1881.

AUGUSTUS HAGHE, eldest son of Louis Haghe, the well-known artist, one of the Past Presidents of the Institute in Water Colours, was born on the 25th of May, 1846. He inherited much of his father's artistic talent. He began his engineering career in the employ of Messrs. Simpson, civil engineer, under whom he served in 1866 and 1867. In the following year he became a pupil of Mr. Joseph Phillips. Hardly had he passed his noviciate, when a severe attack of congestion of the lungs necessitated a complete change of climate; for this purpose he made a voyage to Australia. At the end of 1869 he recommenced work as Superintending Engineer for Mr. Joseph Phillips in connection with a scheme at Hull. From Hull, Mr. Haghe, still in the same employ, went to the Blackwall Docks, and thence to Liverpool and Weymouth. On the completion of the Weymouth Harbour improvement contract, in 1874, he entered the service of Messrs. T. and C. Hawksley, for the purpose of building and erecting a gasholder at Folkestone gas-works. The large reservoirs at the Leicester water-works next claimed his attention, and from there he went to Great

Yarmouth to construct filter-beds at Ormesby Broads. He subsequently built and erected a gas-holder at Newark. In 1883 he went to Scotland with Mr. Joseph Phillips to assist in the erection of the Forth Bridge. Three years later he returned to London completely broken down in health, and from that date to his death, on the 15th of February, 1890, he was almost entirely confined to the house. Mr. Haghe patented an apparatus for the improvement of signals, and a machine for facilitating the erection of long ladders, besides other smaller patents. He was elected an Associate Member of the Institution on the 5th of April, 1887. He was not only a most intelligent, courteous, and amiable man, but as frank in his manner as he was honourable in all his undertakings.

EDWARD CLAUDE HAWKES was born on the 29th of December, 1845, and passed nearly the whole of his life in India. His father, Mr. Francis Hawkes, held an appointment on the Great Indian Peninsula Railway, and mainly under the tuition of this gentleman, Mr. Hawkes was prepared for the engineering profession. In 1864 he obtained an appointment as an Assistant Engineer on the same railway, which post he held till 1872. During this period he assisted in the survey of the line from Sholapur to Hyderabad and Raichur, on the south-east branch line, and on the maintenance and reconstruction of bridges on the Nagpur branch. He next entered the Government service, and was appointed an Assistant Engineer 1st Grade, in the Public Works Department, in December, 1873. During this period, he was engaged on various railways, both open and under construction, but in 1879 he resigned, and in the following year was engaged on the Mormugao Hubli Railway Survey, under the Bombay Government ; and in 1881 was appointed District Engineer, Dacca District, Bengal. He resigned this post in December 1882, when he rejoined the Public Works Department as Temporary Engineer on the Bengal Nagpur Railway Survey, and afterwards on the Benares Cuttack Railway Survey. The Survey of the latter line being closed, he returned to England. In June 1888 he was appointed Resident Engineer on the Tansa Dam under construction for the water supply of Bombay, and he held this appointment at the time of his death. For five months, in 1889, he acted as Deputy Executive Engineer in charge of Water-works, Bombay Municipality.

Mr. Hawkes' services gave uniform satisfaction to the engineers under whom he had served; and his premature death, from an illness incurred in the execution of his duties, was much regretted by his friends and fellow officers in Bombay.

He was elected an Associate Member of the Institution on the 11th of January, 1887. He died on the 14th of February, 1890.

JOHN LYONS began his apprenticeship to engineering as a student of the Thomason College, Roorkee, India. After passing the final examination, he was appointed to the Public Works Department, as an overseer, in charge of a subdivision of 78 miles of grand trunk road for two years. From this appointment, on account of extra qualifications as a surveyor and draughtsman, he was transferred to Rajputana State Railway, where he served five years (from 1869 to 1874), two of which were employed surveying, and preparing plans and sections of different alternative lines, and making working plans and drawings of the approved lines. The last three years of the time he was in charge of a subdivision of 18 miles of the construction of the line, at the completion of which he was transferred to the Northern Bengal State Railway, where he also held charge of a subdivision during the Bengal famine of 1874–5. After a year he was re-transferred to the Rajputana State Railway, and subsequently to the Indus Valley State Railway. He then resigned the Government service, and went to Bombay to engage in private practice. He joined the Bombay Municipality in 1876, and was employed in preparing working drawings for the Tulsi Water-works, which he afterwards helped to carry out by superintending the construction of the service reservoirs at Bombay. He also superintended the pipe-laying, and subsequently took charge of the construction of the New Outlet Works at Vehar Lake. He was transferred to the Bombay Drainage Works in December 1879, but resigned this appointment on the 23rd of February, 1882, to take up a post of Assistant Engineer on the Southern Maratta Railway. He was Assistant Engineer on the survey for the railway from Satara to Kolapore, under Captain Burn-Murdoch, R.E., was afterwards Assistant Engineer on the Bellary-Kistna Railway, and later Assistant Engineer on the Indian Midland Railway. He went to England in 1886 for a short furlough, and on his return set up in private practice at Poona. He, however, met with indifferent success, and had applied for and obtained the post of Secretary to the Municipality of Sukkur, when he was stricken down with heart disease, and died on the 28th of May, 1889.

Mr. Lyons was elected an Associate Member of the Institution on the 4th of May, 1880.

PLOTINO SOARES was born in Brazil on the 9th of July, 1850, He was educated in the Polytechnic School of Rio de Janeiro, where he took a doctor's degree in mathematics, physical and natural sciences. In 1875 he was appointed by the Imperial Government Assistant Engineer, under Dr. Francisco Gomes Collasso, the Chief Engineer for the Exploration of the proposed Government Railway from Cuiaba Alagoinhas, Province of Matto Grosso. He then served as Assistant Engineer on the Rio de Janeiro Water-works for one year, from March 1876, under Mr. Edward Lynch, Chief Engineer, and next became Engineer-in-Chief of the Government surveys of the Alto Rio Doce, in the province of Minas Geraes. Mr. Soares was then appointed Engineer of the First Class in the exploration, and chief of section, on the construction of the Paulo Affonso State Railway (80 miles long), in the province of Alagoas, from June 1878 to November 1880, under Mr. Reginald von Krüger, the Chief Engineer. From June 1881 to April 1882, he was on the contractor's staff of the Minas and Rio Railway (100 miles long), of which Sir James Brunlees, Past President Inst. C.E., was the Consulting Engineer, being concerned in the revised studies of the line. He was subsequently Chief Engineer of the surveys for a proposed railway from Antonina to Assunguy (80 miles long), province of Parana, from September 1882 to April 1883. Dr. Soares then served as contractor's district agent of the Bahia Central Sugar Factories, from June 1883 to May 1884, under Mr. W. Chapman, after which he was for six months District Engineer on the Sergipe Railway (80 miles long), under Mr. W. Rogers. On the 30th of April, 1885, he was appointed Government Fiscal Engineer of the Brazil Great Southern Railway (110 miles long), province of Rio Grande do Sul, which appointment he retained until his death in October 1889.

Previous to commencing his professional career Mr. Soares had been aide-de-camp to a Brazilian general during the Paraguayan War, and was wounded during an engagement; and he afterwards served for more than a year in the works of a firm of mechanical engineers at Glasgow. The combined experience thus obtained served to rub off the pedantry of the college-educated "doctor-engineer" of his native country. Being a man of worth and parts, he became very popular with the English engineers of the Great Southern of Brazil Company to whom he was a good friend, and whom in his character as representative of the government, he saved many of the petty vexations and annoyances to which national jealousy is apt to expose foreigners in South America.

Dr. Plotino Soares was elected an Associate Member on the 1st of December, 1885.

LORD MAGHERAMORNE (Sir James Macnaghten McGarel-Hogg, Bart., K.C.B.,) was born at Calcutta on the 3rd of May, 1823. He was the eldest son of the late Right Hon. Sir James Weir Hogg, formerly Member of Council for India, who for many years represented Beverley and Honiton in Parliament. He was educated at Eton and at Christ Church, Oxford, and in 1843 he entered the 1st Life Guards, becoming Major and Lieutenant-Colonel in 1855, and in 1859 he retired from the service. In politics he was a Conservative, and represented Bath from 1865 to 1868, being selected on the assembling of Parliament in November, 1867, to second the Address. In 1871 he was elected Member for Truro, for which place he sat until 1885. In 1885 he was returned for Hornsey, and retained his seat unopposed in 1886.

He took an active interest for many years in matters of local administration, having been a member of the St. Margaret and St. John Vestry, the Westminster District Board of Works, and subsequently of the Guardian Board and Vestry of St. George, Hanover Square. He was elected in 1867 to represent this latter body upon the Metropolitan Board of Works, and on the death of the Chairman, Sir John Thwaites, in the autumn of 1870, was elected by his colleagues to be the successor of Sir John Thwaites, and to this position he was annually re-elected until the Board itself ceased to exist. On the 5th of August, 1871, he, as Chairman of the Board, laid the foundation stone of the Chelsea Embankment, an improvement originally proposed in 1839, but abandoned for lack of funds, and finally undertaken by the Board of Works in 1868, when the necessary powers were obtained from Parliament. At the opening of the Embankment in 1874 Lieut.-Colonel James Macnaghten Hogg, as he was then called, was created a K.C.B.

On the death of his father in 1876 he succeeded to the baronetcy, and in 1877 assumed by Royal licence the surname of McGarel in addition to and before that of Hogg, and the additional arms of McGarel, in compliance with the will of the late Charles McGarel, Esq., of Magheramorne, by which he inherited the estates.

Magheramorne is, as its name implies, the place of settlement of one of the most famous of the Irish tribes, the Mornes. It is situated on the borders of Lough Larne, which was one of the landing places of the Viking settlers in Ireland. The estate has belonged to various families of distinction, and in 1842 came into the possession of the late Charles McGarel, Esq., after whose death, in 1876, it passed into the hands of the subject of this notice, who, on being elevated to the Peerage in 1887, took his title from his Irish estate.

As Chairman of the Board of Works he was prompt and firm in his decisions, showing himself thoroughly acquainted with the rules of debate, and though a little abrupt at times, was always courteous in his manner to all. That he was popular with his colleagues is shown by his repeated re-election as Chairman for so many years.

Lord Magheramorne died on the 27th of June, 1890, of inflammation of the lungs. He was elected an Associate of the Institution on the 6th of February, 1872.

*** The following deaths, in addition to those included in the foregoing notices, have been made known since the 3rd of June, 1890:—

Members.

OLDHAM, JAMES; *born* 23 June, 1801; *died* 10 June, 1890. (*Congestion of the lungs.*)

STUBBS, WILLIAM HENRY; *born* October, 1847; *died* 23 June, 1890. (*Aneurism of the arteries.*)

WATSON, THOMAS COLCLOUGH; *born* 29 August, 1822; *died* 2 July, 1890. (*Apoplexy.*)

Associate Members.

GORDON, EDWARD FRANCIS; *born* 10 February, 1853; *died* 26 December, 1889. (*Malignant jaundice.*)

HOLTHAM, EDWARD PHILLIPS; *born* 29 August, 1846; *died* 19 April, 1890. (*Pneumonia.*)

PADDON, WILLIAM VYE; *born* 10 July, 1859; *died* 9 July, 1890.

Associate.

MARILLIER, JAMES CONSTANTIEN; *born* 21 June, 1827; *died* 20 June, 1890. (*Inflammation of the lungs.*)

Information respecting the life and works of any of the above is solicited in aid of the preparation of future Obituary Notices.—SEC. INST. C.E., 21 *July*, 1890.

SECT. III.

ABSTRACTS OF PAPERS IN FOREIGN TRANSACTIONS AND PERIODICALS.

The New Levelling Survey of France. By C. JIMELS.

(Le Génie Civil, vol. xvii., 1890, p. 5.)

A general levelling of the territory of France was projected and commenced by Bourdaloue in 1857, but was abandoned in 1864, after the base-lines only were completed. The levelling now in progress was commenced in 1884, a sum of £2,000 per annum having been devoted to it each year since that date.[1] It comprises (1) a new system of base-lines, about 7,500 miles long, following the main lines of railway, whose easy slopes permit very accurate levelling; (2) about 500,000 miles of detail levelling; (3), a series of contour lines.

The new base-lines will also serve to connect the French levels with those of neighbouring countries, and to compare the mean levels of the Mediterranean, the Atlantic, and the English Channel.

The entire expense should not exceed £800,000. The work is under the direction of Mr. C. Lallemand, who has described its progress and methods in a work, "Traité de Nivellement de haute précision," recently published.

The base-lines consist of closed polygons, averaging about 380 miles in perimeter. Fixed bench-marks are placed 500 to 1,000 yards apart. Each section between two bench-marks is levelled in both directions, and one day's work is confined to each section. All reductions and calculations are made at a central office. The bench-marks are of oxidised iron or bronze, and built into the walls of solid buildings. They enclose a tablet describing their position in the survey and the level.

The level used is carried on a spherical bearing, which permits the telescope to be rapidly placed approximately level without moving the legs. By means of reflecting prisms the two ends of the bubble-glass are visible to an observer standing at the eyepiece of the telescope, so that he can verify the accuracy of his adjustment at any moment without leaving his post.

The staff used is the "compensating staff" of Colonel Goulier, which contains a double metallic rule of iron and brass, by which the variation in length of the graduations on the wooden staff can at any moment be observed. This variation is observed three times a day by the leveller, and amounts to some thousandths per cent., having a daily and also a yearly period, and varying from one staff to another. The divisions are in centimetres, 5 millimetres and 2 millimetres. These divisions are not accurately equal, a systematically erroneous division being preferred, in which the law of error is only known at the central office, as thus it becomes more difficult for the operators to attempt to correct any observations the results of which do not sufficiently agree.

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[1] This comparatively unimportant amount appears to be devoted to establishing the fundamental lines of levelling. It will be increased as the financial situation permits, when the work will be pushed in all its departments.

A special and original system of bonuses has been applied to this undertaking by Mr. Lallemand, by which the salary earned increases nearly as the square of the work done. Under this system the cost has diminished from £2 12*s*. 4*d*. per mile in 1884, to £2 2*s*. 1*d*.; while the salary of staff-holders (for instance) has risen from 5*s*. 3*d*. to 9*s*. 6*d*.

About two-thirds of the base-lines are now levelled, and this first part of the work will be finished before three years. The probable error amounts, on the average, to less than $\frac{9}{10}$ millimetre for a distance of 1 kilometre (about 0·057 inch per mile), while in the operations of Bourdalouë it amounted to from 2 to 3 millimetres per kilometre (0·13 to 0·19 inch per mile). The probable error from Marseilles to Lille does not exceed 2 inches. The datum level is the mean sea-level at Marseilles, and a new tidal gauge has been established there, in order to determine the datum anew with great accuracy.

C. F. F.

New Formulas for the Strength of Materials under Bending or Torsional Strain. By L. Rey.

(Memoires et Compte Rendu de la Société des Ingénieurs Civils, Dec. 1889, p. 729.)

The nature of the materials used in construction is generally known only by the results of tensile experiments. The Author considers that the usual formulas for the resistance of elastic bodies to flexure or torsion are misleading when, as is usual, the limit of elasticity and breaking stress of the fibres of the body are supposed to be identical with those of the same fibres when submitted to direct tensile stress. In proof of this a flat bar of spring steel, having a breaking strength of 63 to 66 tons per square inch will stretch about ½ per cent. before fracture by direct tension, while it may be bent without permanent set over a cylinder whose diameter is 170 to 200 times its thickness, which involves an extension on the outside surface of 0·6 per cent., thus showing that the limit of elasticity for flexure is higher than the breaking strength by direct stress. The present Paper is an attempt to find such a relation between the strength of a body to resist flexure or torsion, and its strength under direct tensile stress as will enable the former to be derived from the latter by calculation. The principle on which the Author proceeds is to calculate the work done by a given deformation in the cases to be compared (expressing the work in terms of the maximum longitudinal stress in the case of flexure and torsion), and to equate these expressions. In other words, the assumption is, that if two pieces of similar material are strained, one by direct tension and the other by bending, then, if the work done by the external forces on the two pieces be the same, they are in a similar condition as regards the margin of safety they possess.

The result of the investigation is, that if R_f be the permissible stress per unit of area in the extreme fibres of a beam, and R_a the (known) permissible direct tensile stress in the same fibres, the "coefficient of equivalence"

$$K_f \text{ or } \frac{R_f}{R_a} = v\sqrt{\frac{\Omega}{I}},$$

where v = distance of extreme fibres from neutral axis of beam,

Ω = sectional area,
I = moment of inertia.

The bending moment and working stress in direct tension are connected by the equation—

$$M = R_a + \sqrt{\Omega I},$$

instead of the usual equation—

$$M = R_f \frac{I}{v}.$$

For torsion K_t or $\frac{R_t}{R_a}$ has the same value, only that the moment of inertia I is, of course, about a different axis.

Tables are given of the values of K_f and K_t for various sections, and calculations are made on the above principle of the strength of iron and steel in some usual forms. The examples taken are a saw-blade when bent sideways, the leaf of a laminated spring, a steel rail, a wagon coupling hook, a chain link, a spiral spring. In each case the value of R_f or R_t is known from the stresses or strains which the pieces actually endure in working, and in each case this value is superior to the elastic limit in direct tension, while in some it is superior to the breaking strain. Applying the "coefficient of equivalence" K_f or K_t, R_a is found in each case to lie within the limit of elasticity.

C. F. F.

Construction and Use of Influence-lines of Stress in Solid and in Latticed Arches Hinged at the Springings.

By Bertrand de Fontviolant.

(Le Génie Civil, vol. xvi., 1890, pp. 391 and 406.)

An "influence-line" is a curve which represents by its ordinates the effect produced at a particular part of the span by a single isolated load as it travels over the span. Thus we may draw an influence-line of bending-moments at a particular section, or an influence-line of stresses in a particular bar. In these curves the ordinate at any point is the bending-moment, or stress produced when a unit weight rests at that point, there being no other external forces. These curves are largely discussed in Mr. Maurice Lévy's "Statique Graphique." The present Paper gives a construction for drawing the influence-line of stress in extreme fibres at any section of a solid arch, or the influence-line of stress in any member of a latticed arch, hinged at the springings. The influence-line of horizontal thrust is first to be drawn by the method described in Mr. Lévy's work above mentioned (*i.e.* the curve whose ordinates at any point represent the horizontal thrust in the arch due to a single unit load at that point). The intercepts on vertical lines between this curve and two straight lines, for which a construction is given, are the ordinates of the influence-line required. The two straight lines above mentioned are determined solely by the dimensions of the arch and the position of the fibre or link to which the influence-line required refers. Methods are given of determining from an influence-line drawn in this way what is the most unfavourable position (for the section of a solid arch, or member of a latticed arch to which the influence-line refers) of a given set of loads, such as those of an engine and tender, and the most unfavourable distribution of a given uniformly distributed load.

With a hinge at the crown also, the case is the same, only that the influence-line of horizontal thrust is now statically determinate, and consists of two straight lines.

C. F. F.

Cast-Iron. Strength, Resilience, Tests, and Specifications.

By J. B. Johnson, M. Am. Soc. C.E.

(Transactions of the American Society of Civil Engineers, vol. xxii., 1890, pp. 91–134.)

The St. Louis cast iron varies in tensile strength from 17,000 to 36,000 lbs. per square inch, and a specification of 25,000 lbs. per square inch can be readily filled. The test-specimens should be about $1\frac{1}{8}$ inch in diameter and 14 inches long, the middle part turned down to 1 inch diameter for a length of 8 inches. The total resilience is the work done in resisting distortion up to rupture. Since cast iron usually breaks under some kind of shock, it follows that the resilience is many times more important than the strength. To measure the resilience, a stress strain diagram must be drawn. A number of such diagrams are given, and the Author points out that the diagrams are curved throughout, so that the modulus of elasticity varies with the load applied, the greatest resilience found being 137·5 inch-lbs. per cubic inch. Some tests of cast iron, alone and with silicon added, showed an increase of resilience from 46 to 130 inch-lbs. per cubic inch, and strength from 16,000 to 20,800 lbs. per square inch, due to the addition of silicon. A simple form of level micrometer for measuring extensions is described.

The Author then discusses the well-known discrepancy between the cross-breaking strength of rectangular bars, and the strength calculated by the elastic formula. He shows that three-fourths of the discrepancy may be explained, if the stress at a section is taken to vary according to the law shown in tensile and compressive stress-strain diagrams. The remaining part of the discrepancy he attributes to plane cross sections not remaining plane, and to the existence of initial stress in the bars, compressive on the outside and tensile in the interior. Quite similar explanations, except as to the two latter points, have frequently been given before.

The cross-breaking resilience is shown to be generally one-tenth to one-twelfth of the tension resilience. The cross-breaking resilience can only be obtained by a static test; impact tests cannot be relied on. A small machine for cross-breaking tests is described.

A proposed specification for cast iron is given. The most definite points are that, when tested in tension, the iron shall have a strength of 18,000 to 28,000 lbs. per square inch, according to requirements, and shall elongate four-thousandths of its length.

When broken transversely, and the deflection measured, the half product of breaking-load and deflection shall be 20 to 50, according to requirements.

If static strength is required, the central breaking-load shall not be less than

$$W = \frac{24{,}000\, b\, h^2}{l}.$$

W. C. U.

On Concrete Construction and its Application to the purposes of Fortification. By K. Csongvay de Csegez.

(Mittheilungen über Gegenstände des Artillerie- und Genie-Wesens, 1890, p. 139 *et seq.*)

Concrete has recently been employed in Austria and Hungary for military structures on a very extensive scale, amounting in the aggregate to upwards of 300,000 cubic metres. In the application of this material its relative cost as compared with buildings of brick or masonry must be studied under local conditions. The Author explains the nature, composition, and preparation of beton, the distinguishing characteristics of the materials suitable for its production, and the proportions in which they should be used. He assigns as the amount of beton which, under rules laid down by him, can be mixed and utilized by the hand labour of a party of seven men, a total of 8 cubic metres per day, or 1·14 cubic metre (40·26 cubic feet) per man. If the amount of work to be carried out reaches 4000 cubic metres, and if other conditions mentioned can be complied with, it will be cheaper to substitute mechanical power for hand-labour in mixing. Details of the machinery and plant advocated—a combined mortar mill and cylindrical mixing apparatus—are given in plans, elevation, and section. By the use of these mechanical appliances fifty-five men can produce 77 cubic metres of beton daily = 1·40 cubic metre (49·44 cubic feet) per head.

It is stated with reference to stuccoing concreted surfaces that if for any reason it is impossible to execute this work immediately after the frames are removed from the face of the concrete, it is better to omit it altogether, as otherwise it is very difficult to make the coating adhere to the surface of the beton. In cases where the concreting has to withstand water-pressure, it is expedient to render the surface with a coating consisting of 1 part cement, 1 part sand, and $\frac{1}{4}$ to $\frac{1}{6}$ part of slaked lime.

The Author devotes a section to the fabrication of moulded concrete blocks used in substitution for masonry, and to the formation of monolithic concrete sewers, cess-pits, and channels; this branch of the subject is explained by reference to diagrams. The construction of bomb-proof magazines, chambers, casemates, &c., and the moulding of concrete *in situ*, for various purposes, is similarly described. The manner of draining and ventilating concrete structures is discussed, and in order to guard against internal condensation the use of "cork-tiles" as a wall lining is advocated. The Author explains that the information he is able to furnish has been gathered during the course of the last few years, while carrying out over 100,000 cubic metres of concrete work.

G. R. R.

American Railroad Bridges.

By THEODORE COOPER, M. Am. Soc. C.E.

(Transactions of the American Society of Civil Engineers, vol. xxi. 1889, p. 1.)

The Paper begins with a historical sketch of wooden bridges from the year 1660 to 1844, the date of the patent of the Pratt truss, and of iron bridges, of which the first was built over the Erie Canal at Frankford in 1840, up to the present time.

It reviews the methods of designing and calculating bridges prevalent in America until 1874, and dates from that year a great improvement in those methods. In 1878 the Author drafted the first complete specifications embodying the designing and proportioning the details of the bridges for the Erie Railway, leaving the choice of the general proportions to the contractor. These specifications have been modified from time to time with advancing knowledge. The Author then comes to the question of calculating the strains of bridges from uniform loads representing equivalents of the actual train loads, a method which was abandoned by most engineers after 1874. He gives, on pages 27 and 28, an easy and correct method, simultaneously worked out by R. Escobar and by himself in 1880 for determining the maximum shearing force and moment at any point of a truss from a given train load, which method has not before been published, but which is in general use.

In a chapter on the strength of materials, the Author relates that the extensive tests made during the erection of the St. Louis bridge developed the present proportions of the eye-bar, and the best forms of compression members. Testing full-sized members has become general, as also the acceptance of a contract clause that full-sized members may be tested to destruction, provided that the cost is borne by the purchaser if they satisfy the specifications.

The following subjects, like the previous ones, are treated as short essays: VI. Manufacture of Bridges; VII. Erection of Bridges; VIII. Typical American Railroad Bridges and their relative Merits; IX. Amount and Kind of Bridges on the Railroads of the United States; X. Failures of Bridges.

In the last chapter the Author refers to the "distorted evidence" submitted to a foreign society in a Paper, where the number of failures of bridges in the last ten years is stated to be two hundred and fifty-one, and mentions that the great majority of these bridges were of wood. The failures of bridges in the United States have no bearing upon the relative merit of the different systems of construction, and upon the present American practice. They failed because they were inadequately proportioned, and from want of intelligent care and attention.

The Paper contains numerous Tables and engravings.

M. A. E.

Bridge over the Danube at Cernavoda. By Prof. T. Melan.

(Zeitschrift des österreichischen Ingenieur- und Architekten-Vereins, 1890, p. 32.)

This bridge, for which the contract was let to the Fives-Lille Company, of France, for the sum of £313,490, will be the first bridge over the Danube eastward of Neusatz, near the mouth of the River Theiss. Cernavoda, as well as the Black Sea harbour of Kustenji, are situated in the Dobrudsha, the new Roumanian Province, and both are connected by a railway about 40 miles long. The object of the bridge is to continue this railway westward into Roumania. About 90 feet below low-water limestone rock is found, and the foundations of the piers, whose crown is about 120 feet above low water, go down to the rock. The bridge—a single-line railway bridge—consists of five spans, viz.: 459, 459, 623, 459, and 459 feet. Spans two and four are girders, 55 feet 9 inches deep in the centre, and 105 feet over the piers; cantilevers, 164 feet long, project beyond the piers into the first, third, and fifth spans; there is consequently left in each of these spans an opening of 295 feet, which is bridged over by a semi-parabolic girder, 42 feet 8 inches deep in the middle, and 29 feet 6 inches at the ends. On the two middle piers are fixed bearings, and on the four remaining supports are roller-bearings; a movable bearing is also at one end of the central girder of 295 feet span.

The principle of the Forth Bridge, as described above, was adopted after various other principles had been proposed, and after it had become evident that a clear headway for navigation of about 100 feet throughout was required, and that neither a continuous girder nor an arch bridge would have been practicable. The design is by Mr. Saligny, Engineer to the Roumanian Government.

The material for the girders is basic Martin ingot-iron, of a tensile strength of from 27 to 30 tons, with 16 to 21 per cent. elongation. The strain per square inch from the load is not to exceed 6·36 tons per square inch, and from load- and wind-pressure together 7·64 tons. The wind-pressure is assumed at 50 lbs. per square foot on the bridge without the load, and 37 lbs. on the bridge with the load. The bending-strains in some of the long members arising from their own weight amount to 1·2 ton per square inch in the diagonals, and to 1·22 ton in the flanges, and they have been taken account of in determining the sectional areas. Other conditions of strength, and various conditions as to the manipulation of the material, are of interest.

As in the Forth Bridge, the transverse bracing between the top flanges of the cantilever girders is omitted, and this is described as having several advantages. Readers of the article may form their own opinion with regard to this somewhat intricate but interesting question. The distance between the cross-girders varies between 42 feet 8 inches and 23 feet 8 inches, and they are plate-girders with 5·7 tons strain per square inch. The stringers are 7 feet 10½ inches apart, and upon them an iron trough-flooring is laid. The sleepers, 9½ inches wide and 6 inches deep, lie in the troughs bedded in gravel.

The weight of steel in the superstructure is 3,385 tons; that of the trough-flooring 226 tons, and that of the gravel-packing, sleepers, &c., 325 tons; *i.e.*, 3,936 tons in 2,460 feet, or 1·6 ton per foot lineal. The iron caissons will weigh 887 tons. The four piers will contain 1,214,250 cubic feet of stone masonry, and 13,054 cubic yards of concrete.

M. A. E.

Bridge over the Chasm of the Noce at St. Guistina in South Tyrol. By Professor VON GABRIELY.

(Zeitschrift des österreichischen Ingenieur- und Architekten-Vereines, 1890, p. 71, 4 plates.)

This is a wrought-iron arch bridge of 196 feet 10 inches span between the pivots on the abutments, and a rise of 1 in 6, while the depth at the crown—which is not pivoted—is about 5 feet. The weight of iron is 786 lbs. per foot lineal. Further data for the calculation of strength are:—weight of timber planking, 22 lbs. per superficial foot; width of bridge, 19 feet 8 inches; load when crowded, 94·7 lbs. per superficial foot; limit of weight of vehicle allowed, 10 tons on a wheel-base of 11 feet 6 inches by 4 feet 3 inches; range of temperature, 60° Centigrade; wind-pressure, 51 lbs. and 30½ lbs. per superficial foot, as usual; surface exposed to wind-pressure, 1½ times the surface of the elevation; permissible strain per square inch of iron, 5·1 tons.

On account of the great depth of the chasm, the erection took place by transforming temporarily each half of the arch into a cantilever.

M. A. E.

The Channel Bridge: The Economic Aspect of the Project.

By CH. DE LA COULAGNE.

(Le Génie Civil, vol. xvii., 1890, p. 75.)

This Paper is a *résumé* of documents in support of the proposal for a Channel Bridge which have been submitted to the French Minister of Public Works. The descriptive and technical part of the subject having been previously dealt with,[1] the economic side of it alone is now treated of. According to the estimates of Messrs. Schneider and Co. and Mr. Hersent, approved by Messrs. Fowler and Baker, the cost of the bridge will be £15,000,000 sterling for the piers and £19,000,000 for the superstructure or £34,000,000 in all. The maximum traffic that the bridge could accommodate on one double line is estimated at 3½ millions of passengers and 25 millions of tons of goods per annum.

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[1] "Le Génie Civil," vol. xv. p. 589.

A consideration of the exports and imports of England to and from France, Holland, Belgium and Germany, leads to the conclusion that out of the present total of 14,000,000 tons 5,000,000 might be expected to prefer the railway to marine transport, the traffic in bulkier and less valuable goods not being affected by the bridge. The average value of this 5,000,000 tons is at least £60 per ton, and the average charge it would bear, having regard to the present cost of marine transport with its attendant shore charges, is estimated at an average of 18*s.* 3*d.* per ton.

The number of passengers now crossing the Channel is 600,000 per annum; it is considered a modest estimate to assume 1,000,000 per annum as the passenger traffic on a bridge, and the average fare is placed at 9*s.* 7*d.*

On this basis the receipts from a bridge would be as follows:

		£.
Goods	5,000,000 tons at 18*s.* 3*d.* per ton . . .	4,500,000
Passengers	1,000,000 „ at 9*s.* 7*d.* per head. . .	480,000
	Total . . .	£4,980,000

It is concluded that after a proper allowance for expenses and maintenance the net receipts would be sufficient to make the undertaking remunerative.

C. F. F.

New Experiments on the Flow over Weirs (second paper).[1]

By H. Bazin.

(Annales des Ponts et Chaussées, Jan. 1890, p. 9.)

In a previous Paper the coefficients of discharge were given for free discharge over sharp-edged weirs with vertical face, and the effect of velocity of approach was examined. The Author proceeds to study the form of the water-sheet, extending the research to the case where the face of the weir is inclined.

Let the figure show the sheet of water passing over a weir. Let AH = h, AP = p, CD = e. The discharge per unit-length of weir is $q = m h \sqrt{2 g h}$; where m is the coefficient of discharge. According to a recent investigation of M. Boussinesq—

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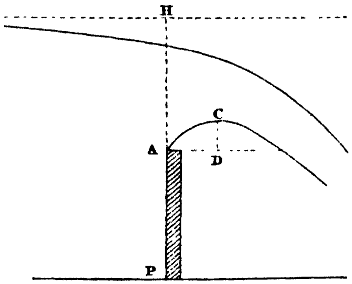
[1] Annales des Ponts et Chaussées, 1888, ii., p. 393.

$$q = 0\cdot5216 \sqrt{2 g} (h - e)^{\frac{3}{2}} = 0\cdot5216 \left(1 - \frac{e}{h}\right.$$

which is equivalent to replacing the total head h by $h - e$ in the ordinary expression; or what is the same thing, giving to m the value $0\cdot5216 \left(1 - \frac{e}{h}\right)^{\frac{3}{2}}$. The ratio $\frac{e}{h}$ is thus a kind of coefficient of contraction.

The Author then describes the means adopted to measure the curve of the upper and under surface of the sheet of water passing over the weir crest, both when the weir had a vertical and when it had an inclined face. A long series of experiments was made with weirs 1·13, 0·5, and 0·35 metre high, and with faces inclined $\frac{1}{1}$, $\frac{3}{2}$, $\frac{3}{1}$ up-stream, and $\frac{3}{1}$, $\frac{3}{2}$, $\frac{1}{1}$ $\frac{1}{2}$, and $\frac{1}{4}$ down-stream.



The ratio of the coefficient to that for a vertical weir increases from the greatest inclination up-stream to an inclination of $\frac{1}{2}$ down-stream, where there is a maximum. For the same head the ratio approaches unity as the height of the weir diminishes. The coefficient m appears to be proportional to $\left(1 - \frac{e}{h}\right)^{\frac{3}{2}}$, except for the inclinations $\frac{1}{2}$ and $\frac{1}{4}$, which no longer correspond to cases of true weirs.

Practical Coefficients.—Since in practice $\frac{e}{h}$ cannot be measured, we must use coefficients of correction, and determine the modulus by which the coefficient for a vertical weir must be multiplied to obtain the coefficient for an inclined weir. The modulus varies appreciably if $\frac{h}{p}$ approaches unity; but omitting that case, which should be avoided, we may admit the following values:—

		Modulus.
Inclination up-stream	$\frac{1}{1}$	0·93
„	$\frac{3}{2}$	0·94
„	$\frac{3}{1}$	0·96
Vertical weir.		1·00
Inclination down-stream	$\frac{3}{1}$	1·04
„	$\frac{3}{2}$	1·07
„	$\frac{1}{1}$	1·10
„	$\frac{1}{2}$	1·12
„	$\frac{1}{4}$	1·09

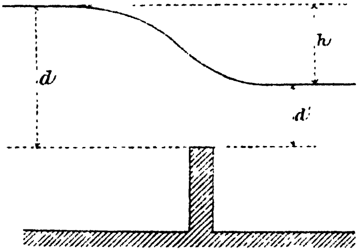
The Author then gives details of the distribution of velocity and pressure in the sheet of water.

W. C. U.

The Problem of the Submerged Weir. By Clemens Herschel.

(Transactions of the American Society of Civil Engineers, vol. xiv., p. 189.)

The problem consists in finding a formula to compute the discharge over a submerged weir. The data for finding such a formula are the experiments in 1848 by Francis; those of Fteley and Stearns in 1887, and those made by Francis in 1883. It seemed to the Author that there might well be a relation between the depth d of water on the weir, and the depth on a weir of free discharge, over which the same volume passed. This may be called the weir of equivalent discharge, and the depth on such a weir, H, is computed by the Francis formula $Q = 3 \cdot 33 \, L H^{\frac{3}{2}}$.



The Author further thought that such a relation might vary only with the variation $\frac{d'}{d}$ of depth of backwater to head on weir.

The whole of the reduced results from the three series of experiments were plotted with values of $\frac{d'}{d}$ as abcissæ and $\frac{H}{d}$ as ordinates. They were found to lie close to a very fair average curve. The following Table gives values of $\frac{H}{d}$ for different values of $\frac{d'}{d}$ measured from the curve. It is from these values easy to compute the discharge from a submerged weir.

With the known values of $\frac{d'}{d}$ find the corresponding value of $\frac{H}{d}$ from the Table or curve. Consequently the head H, on the weir of equivalent discharge can be found, and the discharge computed by the Francis formula.

$\frac{d'}{d}$	$\frac{H}{d}$	$\frac{d'}{d}$	$\frac{H}{d}$
0·00	1·000	0·55	0·871
0·05	1·007	0·60	0·846
0·10	1·005	0·65	0·818
0·15	0·996	0·70	0·787
0·20	0·985	0·75	0·750
0·25	0·972	0·80	0·703
0·30	0·959	0·85	0·644
0·35	0·944	0·90	0·574
0·40	0·929	0·95	0·471
0·45	0·912	1·00	0·000
0·50	0·892	..	..

The Table is abbreviated from that given in the Paper.

W. C. U

Improvement of the Mouths of the Rhone.[1] By ADOLPHE GUÉRARD.

(Rapport de l'Ingénieur en Chef du Service spécial Maritime, dated 31st July, 1888, with two notes, and 7 plates.)

The embankment works carried out at the mouth of the Rhone from 1852 to 1857, stopping up the southern mouths, and directing all the discharge of the river into the eastern outlet, with the object of fixing and deepening the channel over the bar, did not produce the expected results. Since the opening of the Saint Louis Canal, the navigation has deserted the river mouth. The concentration of the discharge into one outlet concentrates the zone of deposit of the sand and silt brought down by the river, which is gradually progressing across the entrance of the Gulf of

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[1] A copy of this Report is in the Library of the Institution.

Foz, and causes an advance of the foreshore along the west side of the gulf. Erosion, on the contrary, has taken place on parts of the southern coast, under the action of the waves, since deposit has been stopped by the closing of the southern outlets, threatening the ruin of the Faraman lighthouse. The advance of the foreshore in the Gulf of Foz seriously impedes the access of vessels to the ports of Bouc and Saint Louis during northerly winds, which are the prevalent winds of that locality, and also endangers the maintenance of the entrances to these ports. Accordingly, some years ago, it was proposed that the old mouths should be re-opened, and the Hannibal embankment prolonged in a southerly direction, so as to divert the deposit of alluvium brought down by the river, to the west of the Gulf of Foz, and thus to arrest the shoaling up of the gulf and the erosion of the southern coast. These proposals were approved by a commission of experts in 1882; but as the works were not carried out, the new Arles and St. Louis Railway Company, and others interested in the locality, have recently strongly insisted on the urgency of re-opening the old mouths, and a careful observation has been made of the changes taking place, and this report presented. The regime of the Rhone delta, given in the first note appended to this report, has already been described in detail in the Author's Paper on the "Mouth of the River Rhone,"[1] published in the Minutes. The shoaling in the Gulf of Foz indicated by observations subsequent to 1876 confirms the views then entertained. In 1872, sailors complained that sailing vessels, and even fishing boats, experienced considerable difficulty in reaching the port of Bouc during northerly winds; and since then the delta has advanced 1,900 feet to the east. Moreover, the port of Saint Louis is increasing in importance, and will require deepening in a few years; whereas even now the entrance-channel and its projecting jetty would need prolonging 2,625 feet towards the east to reach a depth of 26¼ feet; and the depth in this direction is being reduced, on the average, ¾ inch yearly. There is also a tendency towards the formation of an island of deposit between the channel of the large floods and of the average floods, which will gradually deflect the east channel more towards the north, and thus hasten the silting up of the gulf. The above considerations show that the existing condition is perilous, and that an improvement must be effected without further delay. Before the construction of the embankments, the Rhone discharged through six mouths. The largest discharge takes place through the east mouth, amounting to 43½ per cent. of the whole; the discharge through the two northern mouths, Tartane and Pégoulier reached only 3¼ per cent.; whilst, for the three southern mouths, the discharge through the western Piémanson mouth was 14⅛ per cent., through the central Roustan mouth 36 per cent., and through the more eastern Eugene mouth 3⅛ per cent. The two northern outlets may be omitted from consideration, owing to their original very small discharge. The Piémanson mouth is furthest removed from the Gulf of Foz; but the angle, of a little over 100°, which it makes with the line of embankment is unfavourable; and the diversion of an important part of the discharge of the river into this old channel might occasion dangerous perturbations in the currents at the entrance to the Saint Louis Canal. Moreover, the barring of this mouth was effected by large masses of rubble sunk down to a considerable depth, which would necessitate extensive and costly dredging operations for the re-opening of the channel, the total cost of the re-opening being estimated at about £12,000. The Roustan mouth could be much more easily re-opened, as it would be only necessary to lower the portion of the dam formed of rubble to enable the current to scour out the passage. Its southern direction is more favourable for the diversion of a part of the river than the Piémanson branch; and it formerly possessed the largest discharge of the three southern mouths, second only to the east mouth. The Eugene mouth would not cost more to re-open than the Roustan mouth; but it is very near to the east mouth, and its re-opening would impede the diversion of the discharge into the Roustan channel, which it is very important should be as large a proportion as possible, so as to divert the alluvium from the Gulf of Foz to a place where the sea tends to disperse the deposit. The re-opening of the Roustan mouth, besides arresting the progression of the deposit into the Gulf of Foz, will improve the fertility and salubrity of the lands lying outside the embankment, on the right bank below the Piémanson mouth, which, since the diversion of the Rhone, being inundated occasionally only by the sea, have greatly deteriorated in value, and become unhealthy. The only disadvantage which the re-opening of the Roustan mouth will involve is the reduction of the force of the great floods, resulting in their following the more northerly direction now taken by moderate floods, and thus leading the zone of deposit more in the direction of the port of Saint Louis. This must be counteracted by the prolongation of the Hannibal embankment, a necessary work under any circumstances. The prolongation of the Hannibal embankment will necessitate an expenditure of about £68,000, and will occupy a considerable time; whereas the re-opening of the Roustan work can be quickly accomplished at a cost of £4,000. Both works are urgent; but the Roustan mouth can be opened without inconvenience before the prolongation of the embankment is carried out.

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[1] Minutes of Proceedings Inst. C.E., vol. lxxxii. p. 305.

L. V. H.

Suresnes Locks and Weir.[1] By — LUNEAU.

(Annales des Ponts et Chaussées, 6th series, vol. xviii., 1889, p. 49, 11 plates and 23 woodcuts.)

These works, forming part of a scheme for the improvement of the navigation of the Lower Seine, and the increase of its navigable depth to 10½ feet, were commenced in 1880. They comprise a new large lock, on the land side of the old lock, opened in 1882; a small lock, in prolongation of the old lock, opened in 1884, and a new frame weir alongside the locks, in three sections, across the heads of the three channels separated by the islands of Folie and Puteaux, finished in 1885. The large lock has been given an available length of 579¾ feet, the distance between the gates being precisely equal to the distance between the extreme gates of the new and old locks alongside, an entrance width of 39⅓ feet, and a width of 59 feet in the lock-chamber. The small lock has an available length of lock-chamber of 187 feet, with a width like the old lock of 41 feet, the entrance width being 39⅓ feet. The old lock-chamber is 393¾ feet long between the gates. The cost of the locks was as follows:—

	£.
Large lock	69,030
Small lock	37,700
Repairs, &c., of old lock	5,230
Total	£111,960

The navigable pass, adjoining the locks, is 237½ feet in width between the abutments of the weir; whilst the shallower pass adjoining the opposite bank, and the central shallow regulating pass between the islands have each a width of 204⅔ feet between the weir abutments. The navigable pass is provided with fifty-seven frames, whilst the other two have forty-nine frames each; and the apron above the weir is raised about 3¼ feet higher than the floor on which the frames stand, in order to protect the frames when lowered. This apron is sloped gradually down to the floor of the frames, and the apron below is level with this floor, which prevents any accumulation of deposit which could impede the lowering of the frames. The shallower pass on the right bank is closed by hinged curtains, the shallow regulating pass by panels,[2] and the navigable pass by panels and hinged curtains alternately, the panels having been introduced to facilitate the regulation of the discharge in this pass, as panels have proved to be better adapted for this purpose than the hinged curtains. The raising of the curtains from the bottom, with a fall at the weir of 10⅘ feet, produced violent currents and eddies, inconvenient to vessels approaching the locks; and the curtains wore rapidly, and, when two adjacent ones were raised, were liable to touch and impede the rolling-up. There are three tiers of panels for closing the shallow pass, the bottom ones being $3\frac{1}{7}$ inches thick, the central ones $2\frac{1}{3}$ inches, and the top row $1\frac{3}{5}$ inch thick. The panels are each $3\frac{3}{5}$ feet high, and 4 feet wide. The closing of the remaining height of $1\frac{1}{5}$ inch, to reach the full water-level, is effected by a lath, $1\frac{1}{5}$ inch square in section. Five panels are employed for closing each interval between the frames in the navigable pass, $3\frac{1}{2}$, $3\frac{1}{7}$, $2\frac{3}{4}$, $2\frac{1}{3}$, and $1\frac{3}{5}$ inches thick respectively. The weight of a set of these panels is $6\frac{1}{3}$ cwt., and the cost £12; whereas the weight of a curtain closing a similar opening is $7\frac{1}{7}$ cwt., and the cost £27 8*s*. The water-level just above the weir is not kept at a uniform level, as was the system formerly; but the endeavour is to regulate the discharge so as to maintain a uniform level at Paris. When the river rises, it is only necessary to raise the upper row of panels to the point at which the water-pressure just counterbalances their weight. Then, if the water-level continues to rise, the discharge through the opening is increased, whilst if it falls the panel drops a little, owing to the reduced water-pressure against it. If the river continues to rise, the first row of panels is completely removed, and the second, and so on, when the river is in flood. Finally, the frames are lowered, occupying not more than three hours for a whole pass. On the passing off of the flood, the frames are raised in five hours, and the panels replaced in rows as required to maintain the water-level.

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[1] Minutes of Proceedings Inst. C.E., vol. lxxxiv. p. 232, and plate 2, fig. 5.

[2] *Ibid.*, vol. xliv. p. 268.

The cost of the works was as follows:—

	£
Locks, together with various supplementary works .	168,888
Navigable pass weir	35,231
Regulating weir	19,112
Shallower pass weir	26,756
Protection of banks along the three passes . . .	22,579
Embankments on the islands	4,035
Weir-keepers' houses and warehouse	3,308
Land and superintendence	5,353
Total cost of works	£285,262

The weir of the navigable pass, with frames $19\frac{2}{3}$ feet high, cost £148 per lineal foot; the regulating weir, with frames $13\frac{3}{4}$ feet high, cost £93 per lineal foot; and the shallower pass weir, with frames $14\frac{3}{4}$ feet high, cost £131 per lineal foot.

L. V. H.

The New Maritime Installations of the Port of Antwerp.[1]

By H. DE MATTHYS.

(Annales des Travaux Publics de Belgique, vol. xlvii., 1889, p. 1, 22 plates and 85 woodcuts.)

The main purport of this article is a very detailed and fully illustrated description of the new quay works along the right bank of the Scheldt, in front of Antwerp, with certain accessory works, which have been carried out at the expense of the Belgian Government under the direction of the Author. The article, however, commences with a sketch of the gradual development of Antwerp, from the earliest times at which records of it exist, down to 1870. A description is then given of the state of the port of Antwerp in 1870, followed by an examination of the requirements of such a port to satisfy the wants of commerce, its deficiency in this respect, the necessity of improving the port in order to maintain its commercial position, and the means of effecting this improvement. An account is then given of the proposals contained in the Author's report of 1870; the recommendations of the commissions of 1870 and 1874; the plans prepared; and the several steps which, after two sets of tenders had been sent in by experienced contractors in 1876 and 1877, led eventually to the acceptance, in May 1877, of the tender of Messrs. Couvreux and Hersent. This tender comprised the construction of a quay-wall built on caissons founded by aid of compressed air; an abutment connecting the quay-wall with the right bank; the dredging in the river and removal of a projecting point; the embanking of the portion, behind the quay-wall, reclaimed from the river; and the construction of a barge dock, with its accessories, at a total cost of £1,517,600. The line of the quays was arranged so as to rectify the channel of the Scheldt in front of Antwerp, by extending the quay frontage into the river where it was too wide, and cutting off a projecting portion of the right bank, a little above the entrance to the commercial docks, where the river was unduly narrow; and the depth of this improved channel was regulated by dredging. The total length of quay-wall built was 11,330 feet, its line consisting of four circular arcs joining tangentially at their extremities. The quay-wall rests upon a concrete foundation, 29½ feet in width, which has been carried down from 34½ to 52½ feet below low-water, according to the nature of the material reached by excavation in the river-bed, which had been originally examined by thirteen borings. The wall itself has its base at a uniform level of 26¼ feet below low-water, 23 feet in width; and its top is 20 feet 10 inches above low-water, the rise of tide being 11½ feet. The wall is reduced in thickness, especially above low-water level, by a set of steps built at the back commencing at low-water level, and by a straight batter on the face of 1 in 10 up to 19 feet from the base, and of 1 in 20 above. The main body of the wall is composed of brickwork faced with dressed stone from 3¼ feet below low-water to the top; but the spaces left by the shafts of the caissons in the centre of the wall, and the gaps between the several lengths, due to the unavoidable intervals between the adjacent caissons, were filled with concrete. The contents of the wall, exclusive of foundations, amount to 22¾ cubic yards per lineal yard. Five spurs of rubble stone were deposited in front of a portion of the quay-wall to cause accretion where the depth was excessive, and thus afford increased support to the wall. Fenders protect the face of the quay-wall at intervals of 49 feet; and there are mooring-posts along the quay a similar distance apart. Iron ladders, and numerous mooring cross-bars are also provided along the face of the wall. Two floating landing-stages are placed in recesses in the quay. The general width of the quays is 328 feet, and the area of the quays is 80⅓ acres. Sheds have been erected all along the quays; and lines of way, in connection with the Southern Railway Station, run the whole length of the quays, crossing the entrance lock to the barge dock over two swing-bridges, the total length of lines with sidings being nearly 12½ miles, and the number of turntables two hundred. Hydraulic travelling-cranes are placed along the quays capable of lifting 2½ tons, and having a maximum overhang of 31 feet.

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[1] Minutes of Proceedings Inst. C.E., vol. lvii. p. 376; and vol. lxxiv. p. 312.

The construction of the quay-walls comprises three stages, namely, the foundations, the junction of the several lengths of foundations, and the wall, the two first of which only possess any special interest. The bottomless iron caissons employed for the foundations were for the most part made 82 feet long, 29½ feet wide, and from 8½ to 19⅔ feet high, the working-chamber at the bottom being 6¼ feet high, affording an adequate height for the workmen without endangering the stability of the section during construction. The top of the working-chamber is spanned by twenty-four girders, 2⅓ feet in depth; and it is pierced by five circular openings, four of which have a diameter of 1⅔ foot, and the central one a diameter of 3½ feet, each surmounted by a tube, 2⅓ feet high, closed at the top. The site for the caisson was first levelled; and the caisson was towed out, and straightway sunk when the depth was less than 6½ feet. For depths of from 6½ to 16½ feet, a fixed timber staging was employed for sinking the caisson; and for greater depths than 16½ feet, a floating stage was resorted to. In shallow water, the caisson was sunk at low tide, being guided by piles; and the construction of the wall was commenced on the roof of the working-chamber, water being admitted into the upper part of the caisson through holes in the sides above the roof, during the flood tide, to prevent the rising of the caisson till the weight of the wall was adequate, when the water was let out again at low tide. Where fixed staging was employed, the caisson was suspended by chains or screws from the platform of the stage which fixed its position, and was gradually lowered, as it was loaded, till it rested on the bed of the river. Sometimes the levelling of the site was dispensed with, and the caisson was kept level by placing bags of sand under its cutting edge in the deeper places, or by suspending one portion whilst the other part rested on the higher ground. The floating stage consisted of two rectangular flat-bottomed barges 17 feet wide, 85⅓ feet long, and 8½ feet deep, each carrying iron trelliswork uprights and girders, supporting two floors above the barge, connected overhead, across the interval left between them for the caisson, by iron girders, from the ends of which, on each side, the caisson was suspended, together with the plate-iron superstructure enclosing the space on the top of the roof of the working-chamber, and under the shelter of which the wall was built as the sinking of the caisson proceeded. When the caisson had been stranded in its exact position, the plate-iron casing rising above high tide enabled the wall to be carried up. This casing, or movable cofferdam, was 78¾ feet long, 26¼ feet wide, and 32⅘ feet high, and weighed 190 tons. The compressed air having been introduced into the working-chamber, the excavation inside, and the consequent sinking of the caisson, proceeded. The large or solid materials excavated, such as stones and clay, were removed by raising them through the shafts; but materials easily carried in suspension, such as sand, were expelled into the river from the working-chamber by an ejector. This ejector consisted of a box in which the soft excavated material was placed, and into which water under pressure was admitted; the material with the water passed into a vessel, where a thorough mixing was effected by turning a handle, and then, by opening a tap in a pipe connecting the mixing vessel with the river outside, the mixture was ejected by the pressure of the air in the chamber. By this method about 65 cubic yards could be removed in twenty-four hours, each working-chamber being provided with two ejectors. As soon as the caisson had been sunk to an adequately firm stratum, the working-chamber was filled with concrete through the shafts. The movable cofferdam was then unbolted, and, together with the floating stage (which carried the various machinery, and received the materials required for the work), was removed to erect another section of the wall. The construction of an ordinary section of the wall, founded 34½ feet below low-water, occupied about twenty-eight days, out of which twelve days were required for building the wall on the top of the working-chamber whilst the sinking of the caisson was proceeding, after which only two more days were employed in the sinking; and ten days were taken up in filling the chamber and the cavities left in the wall with concrete. The junction of two adjacent sections of wall below low-water was effected by closing the interval on each face by a wooden panel and depositing concrete in the enclosed space, three rectangular grooves having been left at the ends of the sections, which, being filled with concrete, serve like dowels to connect them together. The panel at the back was first pushed down, the interval between the sections was then cleaned out; the front panel was next put down, and the concrete was finally deposited, the whole operation occupying eight days. The wall above was built by tide-work. Details are given of the quantities of materials employed and work effected, and of the plant used for carrying out the works, and the accidents which occurred during the progress of the works. The works of each section of quay-wall were lighted by electric lamps. Four Jablochkoff lamps illuminated the exterior at night, two on the barges, and two on the centre of the floating stage over the movable cofferdam; whilst the working-chamber was lighted by six incandescent Swan lamps, as the stearine candles used at the commencement vitiated the air with their smoke. The power for this lighting was provided on shore. The total cost of the works executed by Messrs. Couvreux and Hersent was increased by the extra depth of the foundations, and the raising and strengthening of the quay-walls, which cost £175,400, and also by other supplementary works, from £1,517,600, the amount of the original tender, up to £1,744,500. To this must be added £950,000 for purchase of land for widening the quays along the Scheldt, about £200,000 for the southern station, and about £210,000 for the equipment of the quays, and other minor sums, the total expenditure on the new quays amounting to £3,314,300. A sketch is given of the dock extensions carried out simultaneously by the municipality of Antwerp. The latest extensions, decided upon in 1883, and still in progress, are the Africa and America docks, on the site of the old fortifications, having a combined water-area of 50 acres, and 11,150 lineal feet of quays; and they will provide a constant depth of 30 feet of water. The new quay works have caused the flood and ebb currents to run parallel to the walls; and the increased depth obtained in the roadstead has been maintained, so that vessels can navigate and anchor with ease and safety in it at all states of the tide. The normal depth at low-water alongside the quays is 26¼ feet, and at the Rhine quay it reaches 29½ feet. The population of Antwerp rose from 136,000 in 1870, to 200,800 in 1884; and the tonnage of sea-going vessels entering the port increased from 698,839 tons in 1864, to 2,134,162 tons in 1874, and 4,102,063 tons in 1884; whilst the tonnage of inland vessels entering increased from 1,030,785 tons in 1870, to 2,361,129 tons in 1884. These results amply compensate for the expenditure of about £4,837,000 incurred by the State and the municipality of Antwerp, in the last fifteen years, for the improvement of the port.

L. V. H.

Works of Calais Harbour.[1] By G. Richou.

(Le Génie Civil, vol. xv., 1889, pp. 541, 565, and 596; 2 plates and 11 woodcuts.)

The harbour of Calais, with an entrance sheltered more or less from all quarters except north to north-east, and the nearest port to England, was restricted, up to 1875, by the ancient fortifications. The depth of the channel over the bar, maintained by sluicing, varied between zero and 2½ feet at the lowest low water; so that the mail steamers, drawing only 7⅓ feet, and timed not to arrive at low water of spring tides, were sometimes delayed in entering. The new works, opened in 1889, constructed partly on the site of the old fortifications, which have been replaced by fortifications further out, have greatly improved the accommodation of the port. Dredging, combined with sluicing from the old, and from the new sluicing basin established on the beach, having an area of over 220 acres, maintain a channel of not less than 13 feet in depth at the lowest low water, and 30 feet deep at high water of the worst neap tides. The new sluicing basin, excavated to an average level of 16½ feet above the lowest low water, retains a volume of about 2,100,000 cubic yards of water at high spring tides, to be discharged, with a fall of from 14 to 19⅔ feet, in three quarters of an hour, through five openings each 19⅔ feet wide, closed by balanced gates turning on a central axis, with their sills at the low-water level of ordinary spring tides. The discharging channel emerges into the entrance channel below the tidal quay, only 820 feet above the pier heads. The outer or tidal harbour, bordered with quays on the north-east and south-west, has an area of 15 acres, and a mean width of 525 feet; its depth is 13 feet below the lowest low water, except alongside the south-west quay, 787 feet in length, where the depth is to be maintained at 23 feet to enable the largest vessels to remain always afloat. This quay, furnished with sheds and sidings, is intended to enable large foreign transatlantic steamers to touch at Calais, and discharge cargoes in a single tide without entering the docks. The north-east quay, 1,870 feet long, on which the harbour station of the Northern Railway is situated for the mail service between Calais and Dover, has four iron landing stages for accommodating four steamers from 328 to 393 feet long, and 11½ feet draught. Two parallel locks lead into the dock, having their sills 5¾ feet below the lowest low water, and being 69 feet wide and 438 feet long between the gates, and 46 feet wide and 451 feet long, respectively. An intermediate pair of gates in each lock, dividing the lock-chambers into two unequal divisions, affords a saving of water in locking moderate-sized vessels. The lock-gates, swing-bridges across the locks, capstans, and sluice-gates, are worked by hydraulic machinery. The dock has an area of 29⅔ acres, and the bottom is 1⅔ foot below the level of the lock sills; near the locks the dock is widened out to facilitate the movement of vessels. Sheds have been erected on the western quay by the Chamber of Commerce; and all the quays are served by sidings of the Northern Railway Company. The graving dock, 508½ feet long, will admit vessels 492 feet in length; the width and depth of its entrance are the same as those of the large lock, and three large centrifugal pumps are designed to pump it dry in less than three hours. A barge dock of 10 acres, and other minor works have been carried out. The excavations, amounting altogether to about 7,200,000 cubic yards, were effected partly by manual labour and partly by a Couvreux bucket excavator. This machine gave excellent results in the deep extensive excavations of the dock, the greatest daily output reaching 3,140 cubic yards in a period of fourteen hours of work. The cost of the earthwork generally, excavated and deposited to form embankments at an average distance of 4,210 feet, was 8½*d.* per cubic yard. The sheeting, against which the toe of the sea embankment of the sluicing basin was to rest, was driven with so much difficulty in the wet sand of the beach, that the engineers had recourse to the injection of water to assist the driving, with such remarkable results that it was adopted for almost all the timber dams enclosing the foundations. The driving of a panel of sheeting, 8⅕ feet long and 6 feet wide, in the ordinary way required 900 blows on the average of a ram weighing 11⅓ cwts., and occupied on the average 8½ hours; and the thickness of the timber had to be increased from 3⅛ to 4¾ inches, to reduce the fractures. By the injection of water, however, through two 1-inch iron tubes, with india-rubber prolongations, the time occupied in the descent of a panel was reduced to an average of one hour and nine minutes; and the number of blows of the ram, needed merely to overcome the friction between the panels, varied from 0 to 50; whilst the thickness of the sheeting could be reduced to 3⅛ inches.[1] In February 1882, the sea embankment, where it served as a dam during the excavation of the sluicing basin sluices, was breached by the sea at a very high spring tide accompanied by a north-west gale; and the water, after pouring into the excavations of the foundations of the sluices, burst into the sluicing basin, making a breach nearly 330 feet wide and extending down to the lowest water-level. After about three weeks the tide was excluded from the sluicing basin; and the breach was closed by the deposit of 125,000 cubic yards of material about three and a half months after the accident. In reconstructing the embankment, the crest of the pitched slope was kept at the original height of 11½ feet above the highest tides, and the width of the embankment was retained, namely, 115 feet at the site of the breach; but the top of the bank was sloped up 1 in 10, and pitched on the outer 33 feet, and a turfed mound, 5 feet high, was placed about 98 feet back from the crest of the pitched slope as a final obstacle to the sea, whilst a masonry apron at the toe, following the slope of the beach, protects it from undermining; and this embankment has perfectly resisted the worst storms occurring during the highest tides. The deepening of the outer pass by suction dredgers, to a distance of 1,300 feet beyond the jetties, was commenced in June 1881; and from 1883 to 1888, 1,506,700 cubic yards were removed and transported 1 mile away by contract, at 6·7*d.* per cubic yard, making a total, with the previous works, of 1,926,700 cubic yards. The channel was previously narrow and winding, with a depth over the bar of only 1 foot 2 inches below the zero of the charts; whereas now the depth is not less than 13 feet over a wide channel. The maintenance of this depth would require the annual removal of about 222,000 cubic yards, which at the contract price would cost £6,200 a year; but the State hopes to reduce this annual cost to about £3,400 by taking over the work. The five sluice openings of the sluicing basin are separated by piers 11½ feet thick, which, with the sills and aprons, rest upon a foundation of piles and concrete enclosed within sheeting. The cost of this work, with the bridge across it, was £68,000. The foundations of the quay-walls of the outer harbour, in running sand, were effected by sinking masonry wells, rectangular outside and hexagonal inside, the largest external dimensions of these hollow blocks being 26¼ feet square and 28 feet 8½ inches high. They were sunk by scouring the sand from under the block by a continuous powerful current, created by the combined action of a forcing and suction pump, which ejected the mixture of sand and water from the well. The block was sunk from 13 to 16½ feet; and subsequently the well was filled up with concrete. The largest blocks occupied on the average forty-four and a quarter hours in sinking to an average depth of 26¼ feet. The quay-walls of the outer harbour, 2,526 feet in length, cost about £110,000. The 42,997 cubic yards of foundations below the general level of the excavations cost an additional sum of 2*s.* 4*d.* per cubic yard for sinkage, including the removal of the sand. The two locks leading to the dock, resting upon a foundation of concrete averaging 5 feet in thickness enclosed within sheeting, cost £212,000. The concrete foundations of the quay-walls of the dock were laid within sheeting, which, originally estimated at £18,000, was reduced by the application of the injection of water for sinking to £6,400; and the work moreover was carried out in a much shorter time. The quay-walls of the dock and inner basin, having a total available length of 6,070 feet, cost £160,000. The graving dock, together with its sump, has cost £108,000; and the pumping machinery and the hydraulic machinery of the gates of the aqueducts, in course of erection, have been estimated at £19,000.

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[1] Minutes of Proceedings Inst. C.E., vol. lxxi. p. 461.

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[1] Minutes of Proceedings Inst. C.E., vol. liii. p. 312.

L. V. H.

The Bellot Dock at the Port of Havre.[1] By — Desprez.

(Annales des Ponts et Chaussées, 6th series, vol. xvii. 1889, p. 1, 19 plates.)

The new North Dock, now called the Bellot Dock, was partially opened for traffic in 1885, and was completely finished in 1887, the works having occupied seven years. The dock is situated in the Eure creek, to the south of the Tancarville canal, on land reclaimed from the sea, from which it is separated by a masonry sea-wall and a timber staging. Its total length is 3,788 feet, including its entrance to the Eure basin, closed by one pair of gates, 98½ feet wide and 328 feet long, and also a central passage of the same length connecting the two basins into which it is divided, the west basin being 1,476 feet long, and the east basin 1,656 feet long, and both of them 722 feet wide. Swing-bridges afford communication across the entrance and the passage. The total length of quay walls is 8,710 feet, of which 7,808 feet are available for shipping. A channel has been formed at the eastern end of the south quay, through which water can be admitted during flood-tide, so as to reduce the entering currents between the jetties and in the locks. The outer embankment, in continuation of the St. Jean sea-bank, is formed of a masonry and concrete wall on the sea face, having a curved batter from a little above high water of spring-tides to the base, with an earthwork embankment at the back, having an inner slope of 3 to 2, which has now been covered up by the filling for the quays. The wall is 3,280 feet long, and is founded on a layer of concrete 5 feet thick, deposited within sheet-piling below the surface of the beach. In prolongation of the wall a timber staging 1,770 feet long, protecting an earthwork embankment, has been extended to the end of the creek, to provide a place of deposit for the surplus material from the excavations, and afford land for sidings and warehouses. The entrance and the west quay walls of the western basin were excavated in the ordinary way by aid of pumping, whilst the outer embankment for reclaiming the site of the dock was being carried out; and the greater portion of the quay walls of the western basin, for a length of 3,280 feet, were constructed simultaneously, by sinking rubble masonry wells before the exclusion of the tide, as previously described.[1] A cross dam, formed along the line of the division between the two basins, enabled the western basin to be opened earlier. The walls of the eastern basin were constructed in the ordinary way, after the completion of the embankment. The sea-wall was constructed by tide-work, assisted by pumping. The construction of the walls connecting the entrance with the Eure Dock, and the demolition of the Eure dock wall in front of the entrance were effected by means of compressed air. The walls founded with wells, contain 80 cubic yards of masonry and concrete per lineal yard; whilst the other walls contain only 73 cubic yards in the western basin, and 61 cubic yards in the eastern basin. An arched passage has been built near the top of the walls all along, 6½ feet high and 4 feet wide, to receive the hydraulic-pressure pipes. The ordinary walls rest upon a foundation layer of concrete, 3¼ feet thick; and in the case of the walls of the eastern basin, the front of this layer has been carried down 1⅔ feet lower, or about 5 feet below dock-bottom, to allow of a future deepening of the basin. The tidal water is admitted at the eastern end of the dock through four openings, each 23 feet wide, and provided with two iron sluice-gates, placed 8⅓ feet apart, so as to have one in reserve in the event of any accident, and thus always secure the closing of the channel on the ebb tide. The excavation for the dock, amounting altogether to 2,857,800 cubic yards, was rendered more difficult by the layer of silt, from 2⅔ to 5 feet thick, which accumulated in the sheltered area during the progress of the sea embankment. Details are given of the arrangements adopted for effecting the excavation, and also of the swing-bridges, hydraulic machinery, sheds, sidings, and other works. The entrance-gates are of plate-iron, with tubular heel-posts and meeting-posts, and nine vertical ribs for supporting the pressure, connected together by horizontal ribs at the top and bottom, resembling very closely in construction and in size the new iron gates of the Transatlantic entrance to the Eure Dock previously described.[1] Each gate is 54⅙ feet long, 36 feet high, and 4 feet 11 inches wide between the parallel sides till they narrow towards each extremity. The cost of the works was as follows:—

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[1] Minutes of Proceedings Inst. C.E., vol. lxxxi. p. 361.

	£.
Earthwork and masonry	547,429
Portland cement for portions of the walling and concrete	91,066
Paving, pitching, culverts, and river works	76,735
Gates, swing-bridges, sluice-gates, capstans, bollards, &c.	25,824
Hydraulic machinery	17,975
Pumping, supervision and sundries . . .	122,873
Land	66,784
Total cost	£948,686

L. V. H.

The Establishment and Maintenance of Harbours on Sandy Shores.

By Eyriaud des Vergnes.

(Annales des Ponts et Chaussées, 6th series, vol. xvii. 1889, p. 185, 6 plates.)

This elaborate article is divided into four sections, of which the first gives an account of the dredging works conducted inside the port of Dunkirk for maintaining the depth in the basins; the second describes in detail the dredging operations effected outside the port, for the improvement of the entrance channel; the third treats of the conditions of the establishment and maintenance of the entrance to the port, and of the roadstead of Dunkirk, and the farther improvements which might be effected; whilst the fourth deals with the general subject of the establishment and maintenance of open harbours on sandy shores. The subjects of the first two sections have been already briefly described by the Author himself, in the correspondence on the Paper on "Dredging Operations and Appliances,"[1] and also in the Paper "Harbours and Estuaries on Sandy Coasts," and in Messrs. Plocq and Guillain's remarks on that Paper.[2] The dredging operations have proved that it is easy to dredge sand outside the port of Dunkirk, and that from 210,000 to 235,000 cubic yards can be dredged annually by each of the sand-pump dredgers eventually adopted. The dredging was commenced when the channel was encumbered by a bar rising 1½ foot above the zero of the marine charts (lowest low water); and a depth has been obtained by this means of 8⅕ feet below zero over the site of the bar, and 6½ feet in the channel, so that the depth afforded for navigation is 26 feet at springs and 22⅔ feet at neaps. The maintenance of this depth in the entrance channel necessitates the removal annually of 654,000 cubic yards of sand. As the shipping dues collected are only about £23,800, the available expenditure on maintenance is restricted; and it has only been by successive reductions, and by eventually dispensing with a contractor, that the cost of dredging was lowered to 1·45*d.* per cubic yard, making the annual expenditure about £4,000. The cost of the three dredgers employed was £16,600. The physical conditions of the coast, and the successive changes of the beach in the neighbourhood of Dunkirk, are described in detail in the third section; and the modifications in the depths in front of the jetty channel, from 1817 to 1885, are illustrated by numerous charts. After describing the existing defects of the entrance channel, the construction of an entirely new entrance is advocated. This entrance would face due north, instead of going to the northwest like the present jetties. The proposed new jetties would be solid, running out from the shore due north, and placed 1,640 feet apart, so as to afford an ample outer harbour for vessels; and these would converge towards their extremities so as to reduce the width of the entrance to 490 feet, which would be situated in a depth of 26¼ feet below zero.

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[1] Minutes of Proceedings Inst. C.E., vol. xcii. p. 450.

As regards the general question of harbours on sandy shores, three plans have been adopted for their construction in places where no stream of importance flows into the sea, namely: (1) A sheltering breakwater in the open sea, connected with the coast by an open viaduct; (2) Parallel jetties, near together, giving access to a tidal-harbour inland, often combined with sluicing arrangements for maintaining the jetty channel;

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[1] Minutes of Proceedings Inst. C.E., vol. lxxxix. p. 73.

[2] *Ibid.*, vol. lxx. pp. 5 and 72.

and (3) Two piers extending out seawards from the coast at a suitable distance apart, and eventually converging to their pierheads situated in deep water. After describing the harbours of Rosslare, Malamocco, Kurrachee, Ostend, Ymuiden, Madras, Dublin, Kingstown, and Howth, as instances of the above three types, the Author arrives at the following conclusions: (1) The dredging operations at Dunkirk have proved that it is possible, even under distinctly unfavourable conditions, to cope with the accretions of sand in front of a harbour, and that sand can be dredged very cheaply; and that, consequently, dredging is one of the practical means of maintaining the depth of the entrance to a harbour on a sandy shore. (2) In studying the conditions of establishing a harbour on a sandy shore, a special investigation must be made of the relative frequency and force of the wind from the various quarters, and of the waves produced, bearing in mind that the waves are the principal agents in the displacement of sand. (3) The only system which, whilst sheltering vessels in front of a sandy coast, runs no risk of modifying the coast line, is the arrangement of one or more breakwaters in deep water, connected with the land by viaducts with piers far apart. The cost, however, of this system is great, whilst the shelter afforded is relatively small; but if the cost does not preclude its adoption, then the breakwaters should be put in sufficiently deep water to allow of some inevitable shoaling in the sheltered area before maintenance by dredging becomes necessary. (4) The system of the parallel jetties, for forming the entrance channel to a port, is excellent where vast lagoons exist inland, filled and emptied at every tide; for the scour, even with a small rise of tide, maintains the depth of the channel. The jetties must be placed far enough apart not to reduce the influx of the tide; and they must be carried out at right angles to the contour lines of soundings, and extended into a greater depth than it is actually necessary to maintain at the entrance. In the absence of inland lagoons, parallel jetties may be adopted if the slope of the beach is not very flat; but the jetties must be solid, not open, at right angles to the contour lines of soundings, and carried out at least to the depth required for navigation; and the advance of the beach must be arrested by dredging in front of the entrance, or on the side from which the main travel of sand occurs. The maintenance of the jetty channel must be secured by dredging, except when the channel dries at almost every tide, when sluicing may be resorted to. (5) The system of converging piers, enclosing an adequate space, is best suited for the exigencies of navigation, and should be adopted when a new harbour has to be constructed on an open sandy shore. The piers may be commenced at right angles to the coast on a flat beach, but generally it is better for them to converge a little from their commencement, and this is imperative on a steep beach. At a certain distance from the shore, depending on the slope of the beach, the piers must be prolonged by converging arms, forming an angle of at least 45° with the coast line; and the pierheads must be carried into a greater depth than that at which the entrance and outer harbour is to be maintained, the exact lines of the outer ends of the piers being determined by the direction of the worst storms, so as to avoid the production of surf at the entrance. The depth inside the piers must be maintained by dredging; and therefore the sheltered area should not be made larger than necessary, as the cost of maintenance is proportionate to the volume of water entering at each tide. If the advance of the foreshore results from the travel of sand along the coast, it can easily be arrested by dredging. If, however, it results from sand brought in from the sea, the piers must be carried well out into the sea, to delay the period when an extension of the piers will become necessary to preserve the depth of the entrance. (6) All harbours opening on to sandy shores will require extensive dredging operations for their maintenance; and economy in dredging is of vital importance. For this purpose, the works should be carried out by the proprietors of the harbour, without the intervention of a contractor. Special attention should also be given to the selection of a site for the deposit of the dredgings where they shall have no chance of injuring the harbour. Pure sand, raised by sand-pump dredgers, can only be deposited in places out at sea, where there is no danger of its returning to the harbour. The inner dredgings, composed generally of fertilizing silt, may be advantageously employed in raising marshy lands, or in covering the sandy dunes, or the upper part of the beach above high-water mark of spring-tides.

L. V. H.

The Proceedings of the International Congress on the Utilization of Running Water, held in Paris, 1889, compared with the previous Congresses on Inland Navigation, held at Brussels, Vienna, and Frankfort-on-the-Main.[1] By F. BOEMCHES.

(Vienna, May 1890.)

The Author alludes to the universal revival of interest in waterways which has been fostered to a very great extent by these congresses, the first of which was held at Brussels in 1885, followed by that in Vienna in 1886, in Frankfort-on-the-Main 1888, and in Paris 1889.

The principal work of these congresses may be summarised as the discussion of the best means of fully developing inland waterways for navigation, together with the best methods of construction, including locks, &c., and manner of working the traffic.

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[1] The original is in the library of the Inst. C.E.

At Brussels the questions to be dealt with by the meetings were finally reduced to six in number, viz. :—

1. How are large seaports best brought into connection with the interior?
2. What are the conditions warranting the construction of a ship canal?
3. Should the canal be the property of the State and free from toll?
4. What cross-section of canal should be adopted?
5. What is the best method of propelling or towing vessels?
6. What system should be preferred for locks and other details?

The conclusions arrived at were to the following effect, viz. :—

1. A canal should, on the grounds of economy, form a connection between as many harbours as possible.
2. The construction of a canal is warranted when there is a large amount of raw products and heavy materials to be carried long distances; or where, between important centres of production and consumption, some material which, on account of its low value, is not worth transporting by the existing methods, would be so were it possible to carry it at a sufficiently low rate.
3. This question was only partially and indefinitely answered.
4. It is remarked that on most canals,[1] the locks are similar to those in use on the French Canal system. In general the section of the canal is governed by that of the locks, and also by the intended speed of travel.
5. A thorough investigation of this question is suggested, and that experiments should be made with models, &c., with a view to obtaining higher rates of speed.
6. Experience so far points to the lock-chamber as possessing the greatest advantages, and as being the best and simplest form of construction.

The question of other methods, such as elevators, should only be thought of where the difference in level to be dealt with is very considerable.

The somewhat reserved tone of these replies may be accounted for by this being the first congress held, but is unquestionable that to Brussels belongs the credit of having been the means of making generally known the particulars concerning the canal systems of Belgium, Germany, France and Holland, and of discussing questions of details of modern construction and management.

At Vienna it was determined to deal less with matters of detail, and only treat of the main questions concerning inland navigation, and this being partly due to the opinion that in Austria the question of the value of waterways had up to that time received but scant attention. The questions determined upon for discussion were :—

1. The value of waterways in relation to agriculture, &c.
2. The normal section for canals, and the dimensions of locks, &c.

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[1] (?) In Belgium.—D. G.

3. Organization for working the canals.
4. The necessity for sea canals.

The conclusions arrived at were as follows:—

That the commercial value of waterways was such, that it was desirable they should be constructed, where called for, of a character in conformity with the wants of the district, even in places where railways already existed; also that in the interests of this question, the statistics regarding the traffic, &c., of existing inland waterways should be kept in a more perfect manner.

For main canals the normal dimensions should be a depth of 6 feet 7 inches (2 metres), a breadth at bottom of 52 feet 6 inches (16 metres); lock chambers to have a depth of water of 8 feet 3 inches (2·5 metres) over sill, and a net length of 188 feet 8 inches (57·5 metres).

For canalized rivers the foregoing dimensions should be adopted as a minimum.

As regards the working of the traffic, an organization of those employed in working the tug-boats was recommended, they being immediately under state control or otherwise. The construction, without delay, of warehouses on a large scale; also the provision of winter harbours would be most desirable for developing the traffic.

The question of sea canals was adjourned to the next congress.

It may be seen that the proceedings of this congress were an advance upon the preceding one, as regards the recommendation for an increase in the dimensions for main waterways, and the desirability of more perfect statistics.

At the Frankfort-on-the-Main congress, the propositions submitted were:—

1. The need for complete statistics regarding inland navigation traffic.
2. The improvement of river navigation.
3. The best form of vessel and method of propulsion, or towage, adapted for the more important waterways.
4. Authorization of sea canals for connection with the interior.
5. The benefits accruing to agriculture by making rivers navigable, and the construction of navigation canals.
6. The improvement and maintenance of navigation at the mouths of rivers.

The subjects it will be seen were most of them similar to those already suggested at the previous conferences, the discussion upon which had either only been commenced, or was still far from complete. Two of the questions, however, were new, viz., the benefit of regulated waterways to the land interest, and the improvement of the mouths of rivers.

As regards the benefit to land-interests, due to canalization and river improvements, in addition to the facilities for transporting raw products and manufactures, there is a very great advantage in a diminution of the risk of injury by floods, and the formation of ice dams, and a consequent increase in the value of the land of the district.

With respect to ship canals, the opinion was that marine navigation should be enabled to penetrate as far as possible inland.

It was recommended that statistics should be furnished, giving in addition to a description of each waterway, and the character of the vessels navigating it, the traffic upon the same in ton-kilometres, and that for assisting in carrying out this recommendation a commission should be appointed, constituted by a representative from each state taking part in the congress. This commission was formed by a dozen members, representing the various nationalities, who met at Berlin in May 1889, and have prepared a Report dated 20th of February last, to be presented at the fourth congress on inland navigation, which was held at Manchester in July of this year. It comprises :—

1st. The inland navigation statistics for, *a.* the various navigations; *b.* the vessels; *c.* the traffic; *d.* the casualties, and recommends—

2nd. The publication of the statistical documents.

3rd. The nomination of an international standing commission for the inland navigation congresses, for the purpose of carrying out the resolutions of the congress for the time being; of preparing the preliminaries for each congress in advance, of placing itself in communication with the various government and other authorities, and of reporting upon what has been done in the interval since the preceding congress, &c.

4th. The institution of a fund for the defrayment of the increased expenses of the commission.

That such a commission should have been appointed seems to be a matter of the utmost importance, in that, by introducing system and method into the work of these congresses the value of their proceedings will be considerably enhanced.

Regarding the second proposition discussed at the Frankfort-on-the-Main congress, viz., the improvement of rivers for navigation purposes, it was the opinion that an extension of such works was needed in view of the increasing traffic, and of general interests; that therefore the capabilities for development for navigation purposes of each river should be ascertained by hydrotechnical investigation, so as to fix to what should be the normal breadth of the stream, and also to ascertain by experiment the character of the works best adapted to the situation.

As regards the vessels, the view was expressed, that under the direction of the authorities interested in inland navigation, experiments should be conducted to ascertain the most suitable form and size of vessels, and the mode of towage or propulsion which would best fulfil the three desiderata of speed, regularity and economy.

So far as the Author is aware, Prussia is the only state which has up to the present practically interested itself in these latter questions, having carried out experiments on the Oder, the Oder-Spree Canal, and the Spree, the result of which is not yet published.

At the fourth congress to be held at Manchester in July 1890 there would be, in addition to the report of the commission already alluded to, two questions remaining over from the Frankfort-on-the-Main congress for discussion, viz., *a*. The question of rates upon rivers and ship-canals, and *b*. As to the tariff and classification of goods upon inland canals.

As regards the progressive work of the three congresses, the advance made was very marked, and at Frankfort on-the-Main the statistics regarding traffic, particulars as to the navigation of various European rivers, types of vessels and other data, including models of the Louvière (Belgium), and Fontinettes (France) elevators, both completed in 1888, were of great interest.

The remainder of the Author's Paper is devoted to the proceedings of the congress held in Paris in 1889, which was much more extended in its scope than either of the foregoing; and embraced eight main propositions, of which four related to the utilization of water in regard to manufactures, supply, drainage, &c., and the remainder concerning the interests of navigation.

These subjects were far from exhausted at the close of the congress, which lasted six days, and included excursions to works of interest, an account of these latter being dealt with by the Author in his conclusion.

The following were the propositions submitted for discussion, regarding the interests of navigation, viz.:—

1. The regulation of navigable rivers with shifting beds.
2. The canalization of rivers, and the various systems of adjustable weirs.
3. The best system of propelling or towing vessels on canals and canalized rivers.
4. Hydraulic elevators and inclined planes for vessels.

The first two questions regarding the improvement of rivers were in continuation of the discussion on the same subject, which had taken place at Frankfort-on-the-Main.

The following is a summary of the opinions expressed:—

1. Experience of the navigable rivers of Europe tends to show that the regulation of a stream with a shifting bed can only be secured by works having the object, firstly of establishing the banks of a channel of the requisite normal section, and secondly by securing the bed of that channel. The solution of this problem is, in the case of the Rhone, made more difficult, by the rapidity of the stream and the consequent shifting of the Alpine boulders and pebbles composing the bed. Therefore the works should be executed in two periods, the first object being to confine the stream to a channel of the proper breadth by cutting off backwaters, strengthening the banks and the construction of parallel works, followed by the second process of securing the river-bed by crib-work, &c., the result being the creation of a channel of constant width, independent of the water-level.

2. The method of canalizing rivers must depend upon a variety

of conditions, including the minimum depth of water, cost of works, and lastly whether it is done at the expense of the state or private enterprise. The extension of canalization beyond a certain point must be governed by considerations of the character of the works in conjunction with the probable traffic, and where these are unpromising a junction with some other river-course should be made, if practicable.

A depth of 6 feet 7 inches is considered sufficient even for international connecting waterways, and those rivers are recognized as the most favourable for development, where, as the traffic increases, it is possible to increase the depth of the channel; such as is the case with the Seine, which has been deepened from 5 feet 3 inches in the year 1866 to a present depth of 10 feet 6 inches; the Rhine from 6 feet 7 inches to 7 feet 10 inches, and the Aire and Calder navigation, formerly having a depth of 3 feet 6 inches, has been increased to 7 feet 2 inches, and finally to 9 feet.

As to the incidence of the cost of canalization works, the question as regards France is answered by the statistics relating to the inland navigation of that country, which show that the main traffic is upon the Seine, and on those canals connecting Paris with the north-east coal-fields and with the northern ports. The other lines of inland navigation upon which the traffic is important are the Marne Parallel Canal, the Central Canal, and the Bercy Canal, all running in the direction of Paris. On three-fourths of the whole length of French canals the total traffic per annum is less than 100,000 tons (*sic*), and more than 2,000,000 is due to a length of 26 miles. A length of 135 miles yields a traffic of 1,000,000 to 2,000,000, and another of 203 miles from 500,000 to 1,000,000 tons, and these waterways are free of toll. Where such conditions exist works of this character should therefore be carried out by the state, as three-fourths of the system would yield no profit to private enterprise.

A comparison between canalization and regulation, in the treatment of rivers, leads to the opinion that, in dealing with a stream, a particular method is not applicable to the whole of its course, and that whereas canalization by means of movable weirs is generally suitable for the middle stretches, for the upper stretches a side canal would probably be best, and for the lower stretches the ordinary regulation method.

Questions 3 and 4 refer to the best system of actuating vessels, and mode of transferring them from one canal pond to another at different level.

The first of these propositions, although taking almost the foremost place among the questions discussed at this congress, had already excited considerable interest in France, and the problems to be solved had been stated by the Government in the following terms, viz.:—What arrangement of mechanism would, with despatch and economy, best allow of river and canal traffic being carried on by a system providing traction for each boat, and such that the motive force should be constantly at the disposition of any one boat, without the necessity for waiting until a certain number of boats should have assembled, sufficient for a train.

The solution of this problem was best effected by the Lévy and by the Oriolle systems, which have been already described.[1]

It is intended to lay with the Lévy system a length of 87 miles on the Etrun-Janville Canal, connecting France and Belgium. The traffic on this section is 3,200,000 tons per annum, and it is proposed to place the stationary engines (each of 100 to 120 HP.) in pairs, along the canal, at distances of 17·4 miles apart. The wire-rope is about $1\frac{3}{16}$ inch diameter, and weighs $7\frac{1}{2}$ lbs. per yard. The speed will be 2·30 feet per second, and the guide rollers are to be placed, as a rule, 82 yards apart. The cost is estimated at £1,148 per mile, of which £382 is for the rope, £511 for the guide rollers, &c., and £255 for the engines. The cost of working is estimated at £357 per mile. The cost accordingly per ton-mile will be 0·0115 penny, or £2 13*s.* for a vessel of 270 tons towed from Etrun to Janville and returning empty. The towage by horses of such a vessel on this length, at present, would be £6 6*s.* 6*d.*, so that there would be a saving of £2 13*s.* 6*d.*, or 58 per cent., as estimated by the inventor. The conclusion of the congress upon this question was therefore to recommend the adoption of the endless rope on canals where the traffic was considerable, the method of horse-towage being considered as still the best where the former system was impracticable.

It may be mentioned that the Prussian public works department have in the estimates for 1890–91 put down a sum of £6,000 (120,000 marks) for experiments to be made on the Oder-Spree Canal (between the Seddin-See and the Fürstenwalde) with alternative systems of working traffic, viz., one with the endless rope, and the other with small tow-locomotives on tracks running alongside the canal. These experiments are in the interests of the Dortmund-Ems Canal now being constructed.

4. As to the means of transferring vessels from one canal pond to another at a different level, the ordinary lock is not only expensive to construct, and the consumption of water considerable, but a height of from 13 to 15 feet is the greatest amount that can practically be dealt with, and lastly, it is frequently the cause of much delay. The ideal canal of the future is one in which the changes of level should be confined to as few points as possible, leaving long intervening stretches of water workable by steam-tugs or endless rope. For this purpose hydraulic elevators, such as those erected at Louvière and Fontinettes are recommended. The construction of such elevators is very costly, but may be reduced by the substitution of steel for iron and modifications of the details.

The cost of the Louvière elevator, where vessels of 400 tons are

[1] Minutes of Proceedings Inst. C.E., vol. xcix. p. 426.

lifted a height of 50 feet 6 inches, was £56,200, and of the Fontinettes elevator, where vessels of 300 tons are lifted 43 feet, was £74,800, in which case the bad foundations and high price of iron increased the cost. The question of simplifying and improving the details and working of elevators was made the subject of discussion, more especially by Mr. Cadart (Rouen) and Mr. Dufourny (Brussels), whose views on the subject are given. The inclined plane is only casually mentioned by the Author. The second part of the programme of the Paris congress, regarding the utilization of water in respect to manufactures, supply, drainage, &c., was divided into four heads, viz.:—

1. The future of irrigation canals.
2. The use of river-water for town supply.
3. The utilization of water-courses for manufacturing and agricultural purposes.
4. The extension of the use of artesian wells.

In reference to the first proposition, it was the opinion that the state should foster the extension of irrigation works generally; and as regards the use of river-water for town supply, that where the yield from wells is deficient, it is desirable to use river-water where the purity of the latter can be ensured. A constant supply to the houses is most desirable where possible, and intermittent supply should, where practicable, be done away with. The adoption of measurement by meter is recommended as being the fairest method in the interests of both the supplier and the supplied, for governing the charge for consumption.

In treating of the utilization of watercourses for manufacturing and agricultural purposes, each particular case must be treated on its merits, having regard to fisheries that may exist, to the agricultural and manufacturing requirements of the district, not forgetting navigation, the object being to obtain the very largest amount of service, and to utilize to the utmost extent every drop of water-flow. The Author refers to the Nile dams, and to the project of a vast reservoir that might be constructed in the plain of Kom Ombo, whereby 988,400 acres of desert might be brought under cultivation.

Artesian wells are regarded as important factors in the development of the prosperity of a country, and reference is made to the desert of Sahara and the possibilities of rendering fertile enormous tracts in the neighbourhood of Algiers and Tunis.

A list is given of seven essays by various authors, forwarded to the congress upon questions of—

1. The rights, individual and collective, to watercourses, by Beaurin-Gressier.
2. The regulation of rivers in Austria-Hungary as regards navigation, by F. Bömches.
3. The Danube regulation works at Belgrade, by Mr. Knezevich.
4. Seine improvement works, by Mr. Caméré.
5. Utilization of waterways in China, by Mr. Tscheng-Ki-Tong.

6. Project for a dam on the Upper Nile, by Count de la Motte.
7. Utilization of the waters of the Rhone at Geneva, by Mr. Turrettini.

There were also read papers of considerable interest upon the future of irrigation works, as regards Europe, Egypt, India and America, by Mr. Cotard; The utilization of river-water for water-supplies, by Mr. W. H. Lindley;[1] Canalization of rivers and the test system of adjustable weirs, by Mr. Vernon-Harcourt; On the best method of towage of vessels on canals, by Mr. Derome; Details and practical results of the Belgian elevator, by Mr. Dufourny; Canal from Havre to Tancarville, by Mr. Widmer; Extension of the use of artesian wells, by Mr. Caméré, &c.

Excursions were made by the members of the congress to—

1. The St. Denis canal.
2. The St. Maurice and St. Quentin canals.
3. Weir arrangements at Port-à-l'Anglais and at Surennes.
4. Elevator at Les Fontinettes.
5. The Seine regulation works between Vernon and Rouen, down stream, and from Havre through the Seine estuary back to Rouen, up-stream.
6. The harbours of Rouen and Havre.

A description of these visits is given by the Author along with a considerable amount of data relating thereto.

D. G.

The Utilization of River-Water for Water-Supplies.

By W. H. Lindley, M. Inst. C.E.

(Deutsche Vierteljahrsschrift für öffentliche Gesundheitspflege, 1890, p. 191.)

The duties of streams and watercourses—the water-supply, and the drainage of the districts traversed by them—are pointed out. In the performance of the latter service, even when the country they pass through is purely agricultural, they can scarcely escape pollution, when the water comes to be regarded from the urban-supply point of view, and this contamination becomes all the more serious when the river-water receives the drainage from towns and factories. The Author states that the ideal method of employing the rainfall would be the interception of such water as had passed into the earth, during the course of its underground flow. Subsoil water, in the shape of natural springs, has always been preferentially selected as a source of supply. As far as quality is concerned, water of this kind when pure must be placed in the front rank—though in quantity, when the demands of a large population have to be studied, it is liable to be deficient. The facts which must be considered as bearing upon the durability and productiveness of springs are discussed. As instances of towns supplied with spring-water ample in volume and excellent in quality, the Author names Dresden, Cologne, and, to a large extent, Düsseldorf, Elberfeld, and Bonn. An ideal supply from the qualitative and the quantitative aspects would be one drawn from natural and artificial sources, or a combined supply of natural with artificially-obtained water, in which the regular flow of the former should, when necessary, be supplemented by deep-well water, drawn from the great natural reservoir in the interior of the earth. The town of Frankfort-on-the-Main possesses a water-supply of this character, the excess of water needed in the summer months being artificially procured, to fill the deficit in the amount obtained from springs. The relative volumes of each kind of water used throughout the year are shown by means of a graphic diagram. When spring-water fails, the next best source of supply will be the surface-water falling in upland regions, which may require to be collected and impounded. Water-supply is daily becoming a question of greater difficulty, and in some instances pure water has to be brought from considerable distances (200 kilometres and upwards); 100,000 cubic metres per day can be conveyed in cast-iron pipes, for the same price per cubic metre, three times as far as 10,000 cubic metres. The cost of conveying water 200 kilometres (124¼ miles) is 4 pfennige per cubic metre; the average price of service-water being from 8 to 15 pfennige per cubic metre (4·3*d.* to 8·1*d.* per 1,000 gallons).

In default both of spring-water and of that from mountainous districts, the water from the valleys and lowlands may be used; and the Author states that an unnecessary degree of prejudice is often expressed against river-water. The cause of objection to such water is its relatively high temperature in summer, but he shows that the passage through the mains to the consumer serves to equalise the primary difference in temperature between spring- and river-water. The advantages and disadvantages of a double service of drinking-water and unfiltered water are examined.

Having decided upon the lower reaches of a river as the source of supply, much depends upon the selection of the spot whence the water is taken, with respect to impurities washed from the river-bank, sand, anchor-ice, &c. Nearly all such water will need purification, and this can be accomplished by resorting to (*a.*) natural filtration; or (*b.*), deposition in reservoirs, and artificial filtration. Each system of treatment is considered in detail. The Author lays down as a principle that "the judicious combination of subsidence and filtration is, as a rule, the best mode of cleansing river-water for the purpose of an urban supply." The two plans of working the filter-beds, either on the intermittent or the continuous system, are examined, and the experience gained at the Frankfort waterworks is set forth. Detail sections are given of these covered reservoirs, and the mode of working them is explained.

With reference to filtration, the artificial sand-filter introduced the Chelsea works by James Simpson, in 1839, is stated to be the safest method. In working a filter-bed, three essential conditions are needed: Slow working-speed; evenness of action over its entire area, and regularity of discharge. Each of these conditions is discussed at length.

By reference to a diagram, the automatic regulator employed at the Warsaw works is explained. Certain minor points—the choice of sand, mode of arranging filtering materials, &c.—are likewise examined. The size of the filter-beds in the London works varies from 1,500 to 4,000 square metres. The sizes of the Berlin covered filter-beds are as follows:—

		Square Metres.
At Stralau (1874)	each	3,200
The first set at Tegel (1884) . . .	„	2,200
The second „ (1887) . . .	„	2,550
The Warsaw covered filter . . .	„	2,100

The relative advantages gained by the use of covered *versus* open reservoirs and filter-beds are considered, and the Author sums up in favour of the use of covered reservoirs. The proper depths for the layers of sand and gravel are determined, and special reference is made to the section of the Warsaw filter-beds, the arrangement of the layers of filtering material therein being shown by a photograph. Under the head of cost, the Author states that the ordinary English open reservoirs and filter-beds cost, with all inlet- and outlet-pipes, apparatus, &c., complete, from 40 to 50 marks per square metre (33*s*. 5*d*. to 41*s*. 9*d*. per superficial yard). A recent careful estimate for a large set of covered filters, extending over an area of 48,000 square metres, worked out at 67 marks per square metre (56*s*. per square yard). Those in Berlin cost from 64 to 72 marks per square metre; the Warsaw covered filter-beds cost 80 marks per square metre of sand-area. Uncovered filter-beds cost approximately two-thirds as much as covered ones. The cost of the latter are set down at about 18 marks per cubic metre of effective capacity (6·1*d*. per cubic foot).

The cost of subsidence in covered reservoirs, inclusive of interest and sinking-fund on the outlay, is from ⅕ to ¼ pfennig per cubic metre (9*s*. 1*d*. to 11*s*. 4*d*. per million gallons). Filtration in covered beds will cost from ¾ to 1¼ pfennig per cubic metre (34*s*. to 56*s*. 8*d*. per million gallons). Of the above sum, from 50 to 60 per cent. represents interest and sinking-fund, and from 40 to 50 per cent. wages and material. The bacteriological result of efficient filtration implies reduction of the germs capable of cultivation in 1 cubic centimetre to as low as from 50 to 150, the number in the mains being 300 at the outside. The Author sums up his observations in a series of twenty-four conclusions.

Tables are appended of the numbers of germs present in the Warsaw and Berlin water before and after filtration for a period extending over several years.

G. R. R.

On the Fresh-water Algæ and their Relation to the Purity of Public Water-Supplies. By GEORGE W. RAFTER.

(Transactions of the American Society of Civil Engineers, vol. xxi., 1889, p. 427.)

The Author points out that the knowledge of the sanitary significance of many of the minute forms of plant-life inhabiting water-supplies is limited, and that much misconception exists concerning the causes of the various bad tastes and odours with and by which potable waters are affected. He states that many kinds of algæ will give rise, under certain conditions, to these objectionable manifestations, and, as the result of experiments conducted by him on at least two hundred species during the past three years, he shows that the organisms capable of injuriously influencing the quality of drinking-water are very varied. The algæ are extremely hardy, having been found in active growth beneath the surface of Hemlock Lake, furnishing the domestic supply to the City of Rochester, New York, at a time when the water was covered with an unbroken sheet of ice from 12 to 15 inches in thickness. Many species flourish in the Arctic regions; and a recent study of the algæ of Nova Zembla shows the existence there of one hundred and seventy-two fresh-water species. Though many kinds are thus hardy, some of them are, at certain stages of their existence, peculiarly sensitive to changes in their environment. Some species only live in waters containing some particular substance in solution, such as sulphur, salt, or iron. Some affect sluggish, stagnant ponds; while others would appear to prefer the purest waters. They are found at all depths below the surface, and some species at considerable distances below the surface of the ground. It may be generally stated that the power possessed by these minute cryptogams, of causing sundry bad tastes and odours, is entirely independent of colour. The colourless, the grass-green, and the bluish-green varieties have each given rise to mischief; and the cause, the Author considers, is to be found in the jelly-, starch-, oil-, or sulphur-producing capacity of these plants. No theory has, so far as he is aware, been yet advanced to account for the various tastes and odours, and he puts this hypothesis forward with some hesitation. Certain algæ—as for example, Cladophora—contain large numbers of starch-grains in connection with the chlorophyl granules. This species, when well advanced in decay, produces a nearly unbearable stench; and when the Author experimented with a solution of starch and water, it was found after several days to give off a quite similar stench. Basing his further experiments on this observation, the Author collected all the species of algæ containing starch he was able to find, and he ascertained that those containing starch-grains, or amylaceous granules in the chlorophyl, are liable, if they develop in quantity, to produce at some stage in their decay an unpleasant smell. Again, other varieties of algæ secrete globules of oil, and these may be the cause of the so-called "cucumber tastes," which have been observed in some supplies, especially after the addition to such water of common salt. There is yet a third series of species sometimes credited with causing trouble, and among these are many forms enclosed in a gelatinous matrix or possessing a more or less gelatinous structure; all these appear likely to produce objectionable tastes. The promoters of ground-water systems have, as a rule, assumed that water so obtained would be free from the nuisance caused by the presence of organised life; but this claim to immunity is fallacious, as is shown by the case of Berlin, where the water drawn from the covered wells near Lake Tegel became polluted by the presence of an alga, the Crenothrix kühniana; and at Jamestown, New York, the well-water seems to have become offensive from a similar cause. Plants of the Crenothrix have been found more than 75 feet below the surface. The Author points out that the Volvox globator, a unicellular alga containing not only jelly, but also starch-grains in the (grass-green) chlorophyl, may also cause a fishy taste and smell, as he has been able successfully to demonstrate in the case of the pollution of the Rochester water-supply in the autumn of 1888. An account is given of this outbreak, and of the means taken to ascertain the cause of the mischief.

In the discussion on this Paper many additional facts are brought forward, and at the Convention of the Society the Author, speaking on the same subject, states that he is of the opinion that in some cases these unpleasant tastes and odours are really the result of the reproductive processes of certain of these organisms; that the troubles occur usually in the summer or autumn, which is the fruiting period with a considerable number of species; and that, while in many instances it is certain that decay is the sole cause of the trouble, in others, owing to the fact that decay follows closely on fruiting, both causes may be concerned in the mischief.

Among preventive measures, aeration, the covering in of the reservoirs, and rapid circulation of the water, are advocated. It is found that in water liable to the growth of algæ, small service-reservoirs, of which the contents are rapidly changed, may prevent their development; in other cases, the introduction of fish has proved to be of benefit. In closing the discussion, the Author cites a case in which undoubted mischief was caused to the water-supply of Rochester, in May 1889, by the presence of diatoms, which were accompanied by a considerable mass of nitrogenous material. He was led to estimate the amount of the jelly due to the growth of these diatoms in the lake and reservoirs as at least 2,000 cubic yards. The most efficient remedy known to the Author for the more serious difficulties of this kind is sand-filtration. The Paper is accompanied by nine plates of photomicrographs of the various algæ.

G. R. R.

Vancouver Waterworks. By H. BADELEY SMITH.

(Transactions of the Canadian Society of Civil Engineers, vol. iii. part 2, p. 315.)

The city of Vancouver, which is the eastern terminus of the Canadian Pacific Railway, with a present population of 10,000 souls, had no existence prior to 1886. It is situated on the south shore of Burrard Inlet, the first harbour of magnitude on the Pacific mainland north of the United States. During the summer of 1886 the Author, acting under the instructions of Mr. Keefer, made detailed surveys of the River Capilano, a stream falling into the Inlet at the First Narrows on the opposite, or north side, over against the western boundary of the present City of Vancouver. This river had been chosen as the source of water-supply for the future town in the entire absence of streams of sufficient importance on the south shore of Burrard Inlet. Its choice involved the laying of mains across the Inlet under conditions never before confronted in the history of hydraulic engineering. The works necessitated, in the first place, the formation of a dam across the River Capilano, which, at the initial point of the system has at the lowest stage of the water a discharge of 440 millions of gallons in twenty-four hours. The site of the dam was 6½ miles up stream, where the river is confined to one channel, and the banks are sufficiently high to admit of its construction. By reference to a plan the Author points out the reasons for the selection of this spot for the dam. The cross-section of the river gave at low-water a current of 4½ feet per second, a width of 100 feet, and an extreme depth of 3 feet, the difference of level between high and low-water being 6 feet. It has since been ascertained that in extreme floods the river may rise over 12 feet above low-water level. The width of the channel in ordinary floods is 210 feet. A detailed drawing is given of the plan and section of the dam. The structure is of continuous cribbing, stone-filled, planked, and sheet-piled. It consists of three principal parts, the north and south abutments and the central tumbling-bay. The reservoir created by the dam contains about 14 million gallons, with an elevation of 483 feet above the lowest depression in the pipe-line. Each section of the dam is described in detail. The clear tumbling-bay is 165 feet wide, supplemented by an additional 14 feet of sluiceway when required; total length of dam from shore to shore 384 feet, breadth of base 41 feet 2 inches, cost $15,039·26 (£3,008). The well-chambers, two in number, are constructed in the north abutment, and by them the water from the reservoir formed by the dam is conveyed into the mains, after passing through a triangular-shaped settling pond, constructed of timbers and cross-ties, bolted to the face of the abutment. The pipe-outlet at the rear of this abutment is a crib-continuation thereof, serving as a protection for the mains against the action of the water flowing over the tumbling-bay. Provision is made for two exit-mains; but at present only one is in use, the other being capped at its lower end, and closed at its mouth by means of its gate in the second well-chamber.

The country traversed by the main was densely wooded, and had to be cleared of timber. From the dam for a distance of 12,716 feet in a down-stream direction the 22-inch riveted mild steel main passes along a gradually descending flat, with a total fall of 164 feet. This flat is a narrow strip of land composed of hardpan and granite boulders, bordering the river at the base of the mountains. At the termination of this flat is a rock wall, through which the river has cut a deep cañon; this it was found necessary to tunnel for a length of 280 feet. Moreover, inasmuch as the hydraulic grade-line of the whole system passes considerably below the floor-level of this tunnel, it was necessary that the main from the dam to the tunnel should be of larger diameter than that from the tunnel to the city.; the section of the lower main from the centre of the tunnel down to ordinary high-water mark of Burrard Inlet, a length of 19,320 feet, is constructed of 16-inch mild steel pipes. At the Inlet this main is divided by a cast-iron Y-breech into two branches of 12 inches diameter. One branch of 12-inch cast-iron pipes has already been laid across the Inlet, and preparations are being made to lay the second. The position of the crossing is explained by a diagram—it is situated at the narrowest part of the channel, where the tidal current runs with the greatest velocity. The maximum depth of water at dead low tide is 56 feet, and the extreme width of the crossing at low tide 1,237 feet. The tidal current runs from 4½ to 9 miles an hour. Between the Inlet and the city is a second crossing of an arm of the sea, known as Coal Harbour.

The reasons for the choice of mild steel in lieu of cast iron for the land mains is fully discussed; relative cost, strength, and weight of each metal being given.

A stipulation in the contract, made with Messrs. Keefer and McGillivray, the builders of the dam, who successfully accomplished the work, was that previous to being placed on the bed of the Inlet the whole length of 1,248 feet was to be jointed, leaded, and made perfectly watertight on dry land, and tested at 300 lbs. per square inch. The mode of jointing and testing is explained. The line of pipes, after having been buoyed, was then placed in position by direct hauling from shore to shore by three 4-inch steel-wire cables, and one 9-inch manilla cable, the aggregate power of the four winding engines being 150 HP. The whole time occupied in hauling the line of pipes across the Inlet on August 28, 1888, was nine hours. It was subsequently ascertained by a diver that the heavy projecting bells of the pipe-joints had scooped a deep groove in the soft sandstone rock forming the bottom of the channel, and the pipes were embedded in a trench they had themselves excavated. The form of flexible joint used (Ward's patent) is shown in a diagram, and details are appended of the flexible cast-iron joint which was fixed on the ends of the steel pipes for the 16-inch mains, and of the Moore and Smith patent joint. A full estimate is given of the quantities and cost of all the work, and tables of the dimensions and weights of the various castings. The Paper is illustrated with six folding diagrams. G. R. R.

The Waters of the Passaic River and its Tributaries.—A Chemical and Sanitary Investigation.

By Henry Wurtz, Ph. D., assisted by Durand Woodman, B.Sc.

(The Engineering and Mining Journal, 1890, pp. 340 *et seq.*)

This Report, made in 1882, is only just published owing to delays caused by legal objections. It forms the basis of a decision by Chancellor McGill, of New Jersey, wherein the chief facts herein set forth are adopted, and which contains also the ruling that "a tideway is State property, to be applied for the common benefit of its riparian population, and that complaints of river pollution, when sustainable, justly apply only to streams above their tidal range." The Author states that his investigations decide the important fact that sewage-polluted streams purify themselves through natural agencies in the course of their flow. The conclusions arrived at are based on analyses covering a period of four months, during the autumn and winter of 1881–2, a season of exceptional character owing to a prolonged drought. There was, in fact, no rainfall which contributed anything appreciable to the flow of the river or its tributaries from early in July until the 29th of December. No ice was formed in the River Passaic until the 3rd of January. The samples were collected between the 7th of September, 1881, and the 7th of January, 1882. The Author affirms that rarely before has an opportunity occurred for the examination of river-pollution during so long a time without disturbance of the uniformity of the stream through natural agencies. The results of the analyses are set forth in eight Tables, which indicate the composition of the water of the Passaic River at various points in its course, that of the Dundee Lake, through which it flows, the Dundee Canal, and a tidal channel beyond, between Passaic and Belleville. The Tables show the total solid residue, the combustible matter, the ash, the sulphuric acid (as SO_3), the chlorine, and the common salt corresponding to the amount of chlorine present. The proportion of ammonia (as such), albumenoid-ammonia, ammonia corresponding to the nitrogen present as nitrates and nitrites, total ammonia, and total nitrogen in the ammonia, are also given. The results are calculated in grains per United States gallon of 58,320 grains, and are compared with those obtained by Dr. Frankland, set forth in the 'Report of the Rivers Pollution Commission,' the total nitrogen having been especially computed for the sake of this comparison.

The various physical conditions of the river, lake, canal, and estuary are described; the bed of the stream is in some places covered with a dense and matted vegetation of weeds, abounding with crustaceans and molluscs. The river-water is alkaline, and comparative analyses prove that in a flow of about 4½ miles from the mouths of the sewers, the total animal matters discharged by a population of fifty thousand persons have been destroyed, the greater part of the effect being due to the last 2½ miles of the current. Even when the stream was frozen over, and somewhat swollen, the same phenomenon was observed. The loss of nitrogenous matter, moreover, is an absolute loss, and no mere transformation; the nitrogen either passes into the air as ammonia, or, if converted into nitrates and nitrites, goes to fertilise the water-weeds. Special attention was directed to the examination of the tidal waters of the Passaic, to which about one-fourth of the analyses pertain, and which are much polluted by the population of Newark. The large amount of salt present in these waters is partly due to sewage contamination, and is partly the result of admixture with oceanic brine, brought up by rising tides. The difficulty of ascertaining the percentages due to each factor has been overcome by the estimation of the sulphuric acid present in ten different samples of Atlantic brine, wherein the mean relative proportion of the amounts of sulphuric acid and common salt was found to be as 1 to 11·3; so that, in a mixture of ocean-brine and river-water, it follows that the sulphuric acid present, over and above that found in the river-water, must, if it be derived from the ocean brine, be accompanied by 11·3 times its amount of salt belonging to the ocean brine, and any excess of common salt over this must have been present in the river-water, where it would indicate previous sewage contamination. A Table had been prepared to show the composition and pollution of the water of the estuary at various states of the tide, based on this ratio. Another key to the solution of this question is furnished by the amount of organic or "volatile and combustible" residues present in the various samples. G. R. R.

On the Strains in Curved Masonry Dams.

By Hubert Vischer and Luther Wagoner.

(Transactions of the Technical Society of the Pacific Coast, 1889, p. 75.)

This Paper was suggested by a discussion, at the American Society of Civil Engineers, of the propriety, or otherwise, of making masonry dams curved in plan. In the Sweetwater dam,[1] which formed the subject of that discussion, the arched form of the dam was relied on to justify a smaller base than would otherwise have been considered prudent. In the course of the discussion however, another curved dam, viz., the Bear Valley dam, was described, the stability of which would have been quite impossible but for its being arched. The Bear Valley dam has a thickness of 3 feet 9 inches at the highest water-level, and a thickness of 8 feet 6 inches at a depth of 43 feet, at which level it stands on a more massive foundation. The radius of the crest is 335 feet, and the dam subtends an angle of 42½° (which gives a length of nearly 250 feet on the crest). This is probably very much the slenderest dam in existence. It is built of granite set in Portland cement, and was finished in 1884, since which period it has had a full head of water against it, with, at times, a sheet of ice 2 feet thick. In the discussion above referred to a suggestion was made (in the words of the present Paper) " to imagine the dam sliced up by vertical radial planes into separate beams of unit width, and to determine the deflections which each such beam would sustain under its portion of the external load, if it were imagined to rest on a layer of springs so adjusted that they may offer the same resistance to compression at all points as would be offered by the arch if compressed to a like degree." The authors first investigate the problem on this assumption. The material is treated as perfectly elastic, but with a modulus along horizontal lines double of that along vertical lines, because the horizontal mortar-joints are much more numerous than the vertical joints, and the assumption is made for purposes of calculation that the compressed arch takes the form of the involute of a circle. The conclusion arrived at on these hypotheses is that the effect of arch resistance (in the Bear Valley dam) would be to reduce the bending-moments at the base of the dam by 19 per cent. The problem is then investigated by assuming the dam divided by horizontal planes into a series of curved layers, each of which is treated as an elastic beam with fixed ends acted on by radial loads. By this means the deflections for successive layers are determined, independently of any assumption as to the form of the curve under strain. The deflections thus formed are somewhat less than those arrived at on the involute hypothesis. The general conclusion arrived at is, that if this method of analysis represented the facts, such tensile stresses would be developed in the masonry as must necessarily have caused very serious cracks, and also that very considerable deflections must exist in the dam. The authors then proceed to investigate the question on a new hypothesis of their own. They neglect the support given by the abutments of the arch, and consider the dam as a beam fixed at its base only; but instead of making its equilibrium depend on the resistance of the vertical section alone, independently of the form in plan, they consider the whole horizontal section of the dam, at any level, as affording the measure of the resistance at that level. The curvature thus increases the moment of inertia of a horizontal section immensely, and the assumption is tantamount to including the versed sine of the curve in the effective base of the dam. The ordinary formulæ for deflection and stresses in elastic beams are applied to this hypothesis (but without any discussion as to how far they are applicable to a beam of curved section whose width is so very great compared to its thickness), with the result that the stresses in the Bear Valley dam are found to be within safe limits. The Bear Valley dam is to be superseded by one of gravity section, but also on a curve, built lower down the valley, and the authors recommend that a series of observations of deflection be made on the present dam at the time when the load is transferred to the new one.

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[1] Transactions of the American Society of Civil Engineers, vol. xix. p. 201; also Minutes of Proceedings Inst. C.E., vol. xcvii. p. 449.

C. F. F.

Experiments relating to the Hydraulics of Fire Streams.

By John R. Freeman.

(Transactions of the American Society of Civil Engineers, vol. xxi., 1889, pp. 303–480.)

This paper contains an account of very extensive and careful experiments on the discharge from fire-engine nozzles, the loss of head in fire hose, the height and distance reached by jets, and other related subjects.

Coefficient of discharge of nozzles.—The pressures were taken from a belt of holes at the base of the play pipe to which the nozzle is screwed, and measured by a mercury column; the water was discharged into a gauging tank. The results are believed to be accurate within 0·5 per cent.

With the most ordinary forms of smooth cone nozzles, with pressures ranging from 15 to 80 lbs. per square inch, the coefficients of discharge were—

Nominal Size.	Mean Coefficient.
¾ inch	0·983
⅞ "	0·982
1 "	0·972
1⅛ "	0·976
1¼ "	0·971

Experiments on a number of nozzles of differing forms gave nearly the same results. With ring nozzles, that is, nozzles having a square edged or sharp edged ring contracting the outlet, the coefficient of discharge is about 0·74. Some experiments on Siamese nozzles are also given.

Loss of pressure by friction in hose.—This was determined for fourteen kinds of 2½-inch hose. The hydrant pressure varied from 15 to 125 lbs. per square inch, and the delivery of the hose from 50 to 325 gallons per minute, corresponding to velocities of 3 to 21 feet per second. The lengths of hose varied from 50 to 325 feet. The loss of head varied nearly as the square of velocity, and almost exactly as the length of hose. The smoothness of the surface of the hose had a very great influence on the loss of pressure. Photographs are given of plaster casts of the interior of the different hose pipes, to indicate the different degrees of roughness. The following figures show the influence of roughness in a few cases.

—	Average Internal Diameter.	Mean Velocity.	Loss of Pressure in 100 feet of Hose if the Hose had been 2½ inches diameter.
		Feet per Second.	Lbs. per Sq. Inch.
A. Solid rubber hose	2·65	13·96	13·4
E. Knit cotton rubber-lined	2·68	13·65	16·0
I. Knit cotton rubber-lined	2·69	13·55	19·4
J. Leather hose	2·80	12·51	21·5
K. Woven cotton rubber-lined	2·53	15·31	25·5
L. Unlined linen	2·60	14·50	33·2

The influence of a small difference of diameter on the loss of pressure, with a given discharge, is greater than is generally appreciated. One eighth of an inch variation of diameter from $2\frac{1}{2}$ inches may cause 25 per cent. variation of pressure with a given number of gallons discharge. The outside diameter of the hose was measured at several places by calipering with the full pressure on, and the thickness of the hose at the same places was afterwards determined. The increase of diameter by pressure varied from 0 to 0·05 inch per 50 lbs. pressure.

Taking the formula of flow to be

where v = velocity in feet per second; r = hydraulic mean radius or $\frac{1}{4}$ diameter; s = virtual slope, the values of n for the above pipes were:—

	$v =$	$n =$
A	12·5 to 20	123·3 to 124
E	7·5 „ 17	111·6 „ 114·8
I	11·5 „ 18	100·1 „ 105·8
K	3·5 „ 19	90·1 „ 94·3
L	3·5 „ 20	73·9 „ 79·6

A sinuous pipe was found to have a resistance about 6 per cent. greater than a straight one. The Author advocates some increase in the size of hose ordinarily used.

Experiments on Jets.—The Author noted the extreme sensitiveness of fire-hose jets to wind. A very slight breeze would cut down the height of the extreme drops by fully 10 per cent. It is difficult to determine what is the height at which the jet may be regarded as a good practical fire stream, the height of the extreme drops being of no practical importance. A Table of heights of jets with various nozzles is given.

Also a Table of the horizontal range of jets at 32° elevation.

Experiments on the best form of nozzle.—It was hoped that some form of nozzle would be found giving a clear polished stream with little spraying for the first quarter of its course. Out of forty nozzles tried not one gave such a stream. For pressures of 5 or 10 lbs. such a stream can be obtained, but not for a good working pressure.

The form of convergence of nozzle makes much less difference than has been supposed. The plain conical nozzle gave a jet on the whole slightly superior to any other form tried; ring nozzles were found to possess no advantage over smooth nozzles, but were slightly inferior in projecting power, and gave greater spraying.

Experiments were made on play pipes with radial diaphragms to prevent rotation of the jet. They did not prove to be very successful.

Practical Tables are added to complete the usefulness of the Paper.

W. C. U.

The Stockholm Sewage and its Influence on the Waters surrounding the Town. By KLAS SONDÉN.

(Gesundheits-Ingenieur, May 1, 1890, p. 281.)

By reference to a block-plan the Author explains the situation of Stockholm on either bank of the Norrström, a short river by which Lake Mälar is discharged into the Baltic. The town waterworks placed 3 kilometres up the Arstawik, a bay opening out of the lake, would, it is shown, be free from all possible sewage pollution, if the direction of the current in the river was always from the lake into the Baltic; but there are occasionally, after a dry summer or autumn, intervals when the level of Lake Mälar sinks to such an extent as to reverse the current in the Norrström, and, under these circumstances, the inrush of salt water might carry with it polluting matters from the sewers, and reach the intake of the waterworks. The fear that evils might thus be caused to the health of the community induced the Chief Medical Officer, Mr. Klas Linroth, to publish during 1888, a treatise showing the possibility of a connection between "Typhoid, Diarrhœa, and the Stockholm drinking-water," with special reference to these so-called "high-water periods." He found that although it might so happen that the sewer contents might reach the water company's intake in a very much diluted state, no spread of infectious diseases can be traced to this cause.

The fact that it was possible that sea-water which had passed the outfalls of most of the sewers might reach the waterworks, led to a further inquiry into the subject before deciding on a system of water-carriage; the points to be considered being set forth under five heads. The town is at present dealt with on the tub-system, and it contained in 1887, 59,734 pan-closets. In the case of 58,826 of these receptacles, an estimate could be formed of the weight of fecal matters annually removed, the contents of the remaining 908 closets had to be estimated. The tubs used are of two sizes, and on the basis of the average weight of excreta removed per tub, the total weight of fecal matters removed in each year is stated at 16,155 tons (15,899·8 tons English). The Author computes the liquids and solids in this bulk of fecal matters, and also the liquids not accounted for, and which he assumes pass into the sewers.

The population of Stockholm was, at the time of the inquiry 219,178. One of the matters to be investigated was the extent to which the adoption of a water-carriage system might aggravate the pollution of the Arstawik, and in order to form a conclusion on this point the present volume of the sewage was measured. The gaugings took place on a main sewer which served about one-eleventh of the population. Tables are given of these measurements of the dry weather flow for each hour of the day and night, while the additional amount on rainy days was observed. The figures thus obtained for the volume of sewage-water per head per diem, viz., 71·31 litres, are in exact conformity with the average quantity of water supplied daily by the water company during the winter months, viz., 71 litres per head. The proportion of the rainfall passing into the sewers varies greatly in accordance with the locality, but as the average of the Author's observations 52·4 per cent. of the rainfall reached the sewers.

The next matter to be determined was the volume of water passing up or down the Norrström at different seasons, and having ascertained the daily volume from gaugings of the flow under each of the four arches of the Norrbro bridge, Mr. O. E. Westin constructed an empiric formula on data thus obtained, and prepared a graphic diagram to suit the same, an illustration of which is appended. The volumes calculated by this means are arranged as a table showing the flow upwards or downwards for each month of the years 1886, 1887, and 1888 in millions of cubic metres. The highest recorded monthly flow seawards being 886 millions of cubic metres in June 1888; the greatest flow in the reverse direction being 201 millions of cubic metres in December 1886. As the total estimated volume of the sewage was but 450,000 cubic metres per month, the proportion of sewage to the total amount of river-water is insignificant. The quantity of solid organic matter now contributed by the sewers to the river-water was, according to the analyses, only 0·5 milligram per litre, and if a water-carriage system were introduced, the total solids would only rise to 2·6 milligrams per litre (0·18 grain per gallon).

The sea-water flowing up the Norrström causes a great rise in the percentage of chlorine present in Lake Mälar. The average amount contained in the water of the lake is 8 milligrams per litre (0·56 grain per gallon). As the salt water runs into the lake it sinks to the bottom, and at the deepest parts some miles inland, the amount of chlorine rises to 2,020 milligrams per litre (141·6 grains per gallon), but very small quantities of the sea-water bearing with it the sewage matters can enter the Arstawik bay. By means of plotting the sections of the lake, the Author estimated the volume of salt water entering the bay, and he states that it would take the water not less than three weeks to reach the waterworks, by which time it would not contain more sewage solids than 0·01 milligram per litre.

G. R. R.

Automatic Flushing Apparatus for Sewers, with discharge in two directions. By A. Schumann, of Worms.

(Gesundheits-Ingenieur, 1890, p. 193.)

The necessity for frequent flushing in the case of sewers with but little fall, or where the sectional area of the flow is relatively small, is now generally recognized. Many contrivances have been introduced for this purpose, and the apparatus designed by the

Author differs from others in that it provides for an independent discharge into two distinct lines of sewers from one flushing-chamber, or in two directions from a flush-tank at a summit-level. The discharge is effected alternately in each direction. The necessary water can be furnished by domestic waste, or from the town supply. The action of the flushing-chamber, which depends upon a tip-tank filled by the overflow water, which tank falls alternately to right or to the left, guided by the motion of a lever arm, and opens the valves leading into each line of sewer, is explained by reference to diagrams.

G. R. R.

The Disinfecting Apparatus of W. Budenberg, of Dortmund.

(Gesundheits-Ingenieur, April 15, 1890, p. 251.)

This apparatus has been tested by Dr. E. Pfuhl, of Berlin, who has published the results of his experiments in the "Military Medical Journal." The action of the apparatus is explained by reference to a diagram. It consists of two parts;—the steam generator, and the disinfecting chamber. The latter, which stands on four legs, is elliptical in section, with flat ends, and is 2·25 metres long, 1·5 metre high, and 0·9 metre in transverse diameter. It contains, therefore, 2·3 cubic metres (81·2 cubic feet) of effective capacity. The chamber is made of galvanized sheet iron, stiffened with six external bands of angle-iron. The ends, one of which is hinged to form a door, are strengthened in a similar way. To avoid any contact between the objects placed in the chamber and the metal work, all the interior surfaces are lined with galvanized wire netting. This precaution is needed because, as all the framework is of single thickness, condensation takes place freely on the internal surfaces and the water collects at the bottom, beneath a galvanized grating, which protects the contents of the chamber from becoming wet. The steam enters at the top, and passes through pipes, slit up throughout their entire length, along the upper part of the chamber. At the lower part of the end wall is a short outlet-pipe closed by a weighted flap-valve. The little upright steam-boiler is provided with a blow-off valve, which limits the internal steam pressure to 0·4 atmosphere. The apparatus is worked as follows:—Steam is admitted slowly into the chamber until the atmospheric air is all expelled through the outlet-pipe at the bottom. This is then closed, and the disinfecting-chamber fills with steam until a pressure therein of 0·2 atmosphere is attained. The valve in the outlet-pipe is weighted so as to lift when the pressure reaches 0·2 atmosphere, and, therefore, when the apparatus is in action the escape of steam and water of condensation exactly balances the amount of steam blown in at the top, and the chamber is worked under this normal pressure, so long as the pressure in the boiler registers 0·4 atmosphere. As the result of his experiments Dr. Pfuhl found that the apparatus was perfectly reliable, and that when tested with anthrax spores, as the most hardy known germs, these were killed without fail in ten minutes after an internal temperature of 100° Centigrade had been registered.

G. R. R.

On the Mode of Estimating the Air Renewal, and on the attainable limits of the same in Enclosed Inhabited Spaces.

By Prof. H. Rietschel, of Berlin.

(Deutsche Vierteljahrsschrift für öffentliche Gesundheitspflege, 1890, p. 225.)

Attention is directed to the two practicable systems of calculating the air-exchange in enclosed spaces; the one being based on the amount of pollution caused by the breath and exhalations of human beings, and the products of combustion evolved by illuminants; the other, on the rise in temperature due to the same causes. With respect to the former system, v. Pettenkofer has laid down that the impurities in the atmosphere are directly proportional to the increase in the amount of carbonic-acid gas therein, and he assigns, as the outside permissible limit for this gas, a volume of 1 per mille. If it be assumed that the average amount of carbonic-acid gas present in pure external air is 0·4 per mille, it follows that not more than 0·6 per mille is the limit allowable in confined spaces for the increase due to such polluting causes. The formula for the hourly rate of air-exchange sanitarily needed may, according to v. Pettenkofer, be thus expressed:—

$$L = \frac{K}{0 \cdot 0006}$$

where L represents the volume of air to be changed hourly, and K the volume in cubic metres of carbonic-acid gas added to the atmosphere hourly. The amount of this gas evolved by a powerful labourer in active work, according to v. Pettenkofer, may be 36·3 litres per hour, giving a value for L of 60·5 cubic metres; a ten-year old boy, as observed by Scharling, produces 10·3 litres, giving in this case a value for L of 17·1 cubic metres. Other methods of ascertaining the increase in carbonic-acid gas are explained by the Author.

Passing on to the second system, based on temperature observations, v. Pettenkofer lays down that a full-grown man, at the temperature of 20° Centigrade (68° Fahrenheit), imparts, at the outside, 100 heat-units hourly to the surrounding air, and a child half this total.

Further, as the combustion of a cubic metre of coal-gas implies the production of 6,000 heat-units, a normal gas-jet burning 0·15 cubic metre (5·3 cubic feet) per hour, may be set down as giving rise to 900 heat-units.

The hourly air-exchange L in cubic metres, expressed in regard to temperature t_0, is thus—

$$L = \frac{W\,(1 + a\,t_0)}{0 \cdot 306\,(t - t_0)}.$$

Here W is the amount of heat imparted to the air hourly, t the temperature of the room ; t_0 the temperature of the external air, and a the coefficient of atmospheric expansion = 0·003665. The Author points out that this formula is unsatisfactory, in that it gives very low values for L if the temperature of the room is, previous to its use, lower than t_0, that of the external air; as also that the first formula, to be satisfactory in application, implies a thorough mixture of the incoming air with the foul air in the apartment. The objections to both systems of estimation are reviewed, and it is shown that the supply of fresh air apparently needed when gas is used, and the second formula is employed, is much larger than the first formula would render necessary. The volumes of air required for sick-rooms, hospitals, gaols, workshops, &c., are discussed, and the best modes of admitting and of exhausting the air are considered. Tables, too long for abstracting, are given of the values of $\frac{L}{J}$, being the number of times the air must be changed hourly, where L is the required volume of fresh air in cubic metres, and J represents the number of cubic metres per head of air contained in the room, the results being calculated for the various multiples of J and L. It is pointed out that, although the pollution of the air is indicated by the carbonic acid, it is not this gas, but the other impurities and exhalations, that either cannot be measured or that cannot as yet be determined, which endanger health. If the pollution be measured by the sense of smell—by no means a bad test—different classes of individuals give rise to foul atmosphere in very various degrees, dependent on habits, clothes, and occupations. As all these existing formulas are based rather on experience and observation, than on strict scientific data, the Author points out that an improved system of calculation is desirable. In the meanwhile, he considers that the subject should be approached from the point of view of temperature, rather than from a study of the contents in carbonic-acid gas.

G. R. R.

The Gas Supply of Berlin.[1] By OTTO LEONHARDT.

(Gesundheits-Ingenieur, May 1, 1890, p. 296.)

The volume of gas used in Berlin, in spite of the extensive introduction of electric lighting, is constantly increasing. This is partly due, perhaps, to the fact of the reduction of 20 per cent. in the price since November 1, 1887, of all gas employed for non-illuminating purposes. The number of consumers which at the close of 1887–88 was 48,460, rose at the end of the following year to 51,047, equivalent to an increase of 5·34 per cent.; the increase calculated on the number of burners in use was, however, 7·46 per cent. Berlin is served by four municipal establishments, and also by the works belonging to the Imperial Continental Gas Association. As the details relating to the latter company's business are not forthcoming, the Author confines himself in the main to the statistics of the four first-mentioned undertakings. Three chief illuminants compete for public favour in Berlin; petroleum, gas, and electricity. A calculation is made of the gas-supplies still extant in places now lighted by electricity, and deducting those in uninhabited houses, or in unused works, the total is shown to be at present 16,577 connections against 16,090 in 1887–88. The total of all electric arc-lights at the end of March 1889 was 3,774, and there were 62,876 incandescence lamps. If it be assumed that each arc-light is equal to six incandescence lamps, and that each incandescence lamp is equivalent to one gas-jet, the existing electric illumination may be taken to represent 85,520 gas flames. The total number of gas-burners in use at the end of 1888–89 was 750,873.

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[1] Minutes of Proceedings Inst. C.E., vol. xcii. p. 470.

The amount of gas used during each quarter of the years 1888–89 and 1887–88 is shown in a tabulated form; the totals for each year being 90,245,000 cubic metres, and 86,346,000 cubic metres respectively. The amount of gas paid for in the year 1888–89 was 84,611,834 cubic metres, leaving 5,633,166 cubic metres unaccounted for. If to the above amounts the gas distributed by the English Company, viz., 32,636,000 cubic metres, be added, a total supply of 117,248,000 cubic metres for the whole city is obtained, and the volume of gas consumed per head per annum appears to be 85·52 cubic metres (3019·7 cubic feet). The greatest quantity of gas made on any one day was on December 19, 1888, when the gas manufactured was 455,800 cubic metres. The smallest quantity made on any one day was on June 26th of that year, when only 103,000 cubic metres were registered.

The proportion of minimum to maximum use of gas in twenty-four hours was the same in both years under consideration, viz., 1 to 5·75. The proportion of largest volume used in twenty-four hours to entire make for the year was as 1 to 191·85, against 1 to 190·57 for the previous year. The greatest hourly maximum to the greatest daily maximum use was in the proportion of 1 to 8·45 against 1 to 8·21 in the former year. Statistics of the coal used, volume of gas obtained in cubic metres per ton of coal, &c., are likewise given.

G. R. R.

Monte Generoso Railway. By L——

(Schweizerische Bauzeitung, 1889, p. 69.)

The construction of this line is now in progress, and about 1·86 mile is formed ready for laying the permanent-way. The original estimate was £80,000, but arrangements were concluded with Mr. Abt, the Engineer, whereby the total capital required was reduced to £72,000 or £12,872 per mile, the whole length being 5 miles.

This outlay is compared with various existing Swiss rack railways, viz., the Rorschach-Heiden costing £24,088 per mile, the Vitznau-Rigi £26,208 per mile, and the Arth-Rigi, £27,385 per mile. The small cost of the line in question is the more noticeable as there are five tunnels upon it, with a total length of 416 yards, and a length of 438 yards of very heavy rock cutting; as compared with the Rorschach-Heiden line, where there is no tunnelling, the Vitznau-Rigi, where there is a tunnel of only 73 yards long, and the Arth-Rigi with three tunnels of together 156 yards. Also on this line the permanent-way is to be entirely either of iron or steel in place of the timber sleepers and iron rails laid upon the other railways mentioned, and the width of formation and depth of ballasting is to be greater.

The low cost of construction is, however, principally due to the careful manner in which the line was originally laid out; to the adoption of the Abt system of rack-rail, admitting of curves of 197 feet radius, the gauge being 2 feet 7½ inches; and lastly the fact that, at the time of ordering the permanent-way and engines, iron was almost at its lowest price. The line, according to the contract, is to be opened by the 1st July, 1890, to the second station and the remaining length on the 1st August. It is to be laid with the compound Abt rack-rail on all lengths where the gradient exceeds 1 in 16⅔. The rolling stock comprises six locomotives, five open passenger cars and three open goods wagons. The locomotives were made at Winterthur and have each a total heating surface of 347 square feet, and weigh 11½ tons when light, or 14¾ tons in running order. Each engine is capable of drawing a passenger car with sixty passengers over the maximum gradient, viz., 1 in 4·54, with curves of 328 feet radius at a speed of 3·1 miles per hour. The mean speed at which such a train (engine and carriage) will travel, up or down, the whole distance, is fixed at 4·35 miles, so that the time occupied in running the length is 1⅓ hour.

The open passenger cars (for midsummer) seat sixty passengers, the closed cars have seats for forty-eight.

The article is illustrated by a plan and longitudinal section of the line.

D. G.

Cost of Working

(Revista de Obras

Year.	Length. — Kilometres (⅝ mile).	Number of Passengers.		Number of Tons.		Distance run by Trains. — Kilometres.	Returns.	
		Total.	Per Kilometre.	Total.	Per Kilometre.		Totals. — Pesetas (9½*d.*)	Per Kilometre. — Pesetas.
1867	5,162	10,230,035	2,003	2,830,900	554	12,690,802	79,619,681	15,593
1868	5,317	10,963,453	2,074	2,568,034	488	12,305,088	76,197,580	14,486
1869	5,426	10,201,270	1,929	2,935,276	555	13,231,839	81,530,774	15,436
1870	5,444	10,976,797	2,048	3,505,603	653	12,511,722	83,366,113	15,559
1871	5,476	10,136,990	1,888	3,737,837	696	13,325,456	90,691,276	16,901
1872	5,501	11,900,176	2,188	4,776,721	878	14,132,997	100,776,985	18,531
1873	5,555	10,523,525	1,945	4,662,033	863	15,388,263	96,341,787	17,844
1874	5,678	10,226,538	2,020	3,495,631	690	15,891,332	92,326,682	18,242
1875	6,011	12,158,647	2,182	3,996,268	717	17,270,872	104,937,646	18,833
1876	6,222	14,378,121	2,403	5,136,628	858	20,712,313	128,171,869	21,422
1877	6,419	13,210,102	2,139	6,548,087	1,060	22,069,866	131,542,866	21,309
1878	6,655	13,287,094	2,093	6,097,132	966	25,739,957	132,346,307	20,855
1879	7,010	14,247,073	2,126	8,823,625	1,316	27,303,035	131,261,988	19,588
1880	7,330	14,813,391	2,111	8,088,177	1,153	29,829,909	151,324,503	21,574
1881	7,317	14,058,913	1,952	8,751,987	1,215	,,	159,222,031	22,114
1882	7,461	16,535,203	2,233	9,398,582	1,269	,,	165,054,813	22,292
1883	7,719	18,366,807	2,405	9,497,872	1,244	,,	173,263,586	22,690
1884	8,253	18,485,370	2,243	9,498,934	1,154	25,098,295	167,651,826	20,380
1885	8,831	17,695,945	2,039	9,221,216	1,063	25,301,170	161,773,595	18,654
1886	8,997	20,191,304	2,267	9,764,962	1,096	25,285,331	174,502,325	19,593
1887	9,209	20,088,928	2,203	9,807,690	1,076	27,648,864	176,337,477	19,339

Spanish Railways.

Publicas, 1890, p. 152.)

Cost.		Net Returns.		Per Train-Kilometre.			Working Cost (Percentage of receipts).
Totals. — Pesetas.	Per Kilometre. — Pesetas.	Totals. — Pesetas.	Per Kilometre. — Pesetas.	Return. — Pts. Cs.	Cost. — Pts. Cs.	Net Return. — Pts. Cs.	
41,997,568	8,225	37,622,113	7,368	6·27	3·30	2·97	52·74
39,779,897	7,563	36,417,683	6,923	6·19	3·23	2·96	52·02
39,139,379	7,401	42,491,395	8,035	6·16	2·95	3·21	47·94
39,291,562	7,333	44,074,551	8,226	6·66	3·14	3·52	47·13
39,089,085	7,285	51,602,191	9,616	6·80	2·93	3·87	43·10
42,173,805	7,884	57,903,180	10,647	7·13	3·03	4·10	42·45
45,073,634	8,348	51,268,153	9,496	6·26	3·35	2·91	46·88
37,723,445	7,453	54,603,237	10,789	5·81	2·37	3·44	40·85
46,775,812	8,395	58,161,834	10,438	6·07	2·70	3·37	44·57
55,361,122	9,253	72,810,747	12,169	6·19	2·67	3·52	43·18
58,250,837	9,436	73,292,029	11,873	5·96	2·63	3·33	44·28
60,045,022	9,462	72,301,285	11,393	5·14	2·33	2·81	45·36
60,672,197	9,054	70,589,791	10,534	4·80	2·22	2·58	46·22
66,545,896	9,487	84,778,607	12,087	5·07	2·23	2·84	43·97
67,525,827	9,378	91,696,204	12,736	,,	,,	,,	42·40
77,000,873	10,340	88,053,910	11,892	,,	,,	,,	46·65
73,892,305	9,667	99,371,281	13,013	,,	,,	,,	42·07[1]
74,866,090	9,101	92,785,736	11,279	6·67	2·98	3·69	44·41
74,679,819	8,611	87,093,776	10,043	6·39	2·95	3·44	46·16
72,007,631	8,085	102,494,694	11,508	6·90	2·84	4·06	41·26
77,648,417	8,516	98,689,060	10,823	6·37	2·44	3·93	44·03[1]

[1] Passenger returns, or per kilometre.

Strengthening the Permanent Way of the Russian Railways.

(Annalen für Gewerbe und Bauwesen, 1890, p. 140.)

The permanent way of most of the Russian railway lines laid down for a comparatively light traffic, which rarely exceeds 30 kilometres (18·6 miles) the hour, is found to be too slight for the recently introduced locomotives of greater weight—the object in Russia, as in other countries, being to attain a higher speed by increased engine-power. The size of the sleepers on main lines, must, in future, in accordance with a decree of October 16, 1889, be not less than 2·7 metres (8 feet 10 inches) in length, 15·5 centimetres (5·9 inches) thick, and 26 centimetres (10·2 inches) wide, the bottom bearing surface being not less than 20 centimetres (7·8 inches) in width. The rails which, under official regulations that have been for some time in force, must be of steel, and which at present vary from 108 to 114 millimetres (4·25 inches to 4·48 inches) in height, and from 26·87 to 29·1 kilograms per metre (54·16 lbs. to 58·66 lbs. per yard) in weight, are in future to be 135 millimetres (5·31 inches) high, and are to weigh 36·27 kilograms per metre (73·12 lbs. per yard). The weights and dimensions of rails now in use in some of the chief countries of Europe, and on certain American lines, are given in a tabulated form.

G. R. R.

Apparatus for Loading Railway Rails. By G. Richou.

(La Nature, March 29, 1890, p. 265.)

As rails increased in length and weight—until now they are 35½ feet long and weigh 80 lbs. per yard, on the Orleans Railway—it became desirable to reduce the cost of handling them, and also the risk of breakage or bending in the process, while increasing the security by obviating the shocks incidental to ordinary manipulation. The apparatus designed and constructed by Mr. Guyenet answers the purpose perfectly. There are now twenty-two of them on the Orleans line, and others are employed on the eastern and western railways of France, and on the western and the northern railways of Spain.

The loading apparatus is erected on the four-wheel platform-trucks specially designed in couples for carrying rails. It is constructed of riveted plate-iron of the form of U, of which the limbs are horizontal. The lower limb is fastened on the floor of the truck; the upper limb carries the lifting tackle, which is on rollers supported on a pair of rails, on which the gear can be traversed across the truck from side to side. It has two drums, on one of which the cable for hauling the rail from the heap is wound by means of a hand-wheel; by the other the rail is lifted. When the rail is lifted to the required height the gear is held in position by a pawl, the winch with its load is moved horizontally to its proper position; the load is then lowered into its resting-place under the control of a brake, and the tackle disengaged. For transporting the rail from the heap to the trucks a couple of rails are laid between them, forming a slight incline, up which the rail is hauled.

The work of loading or unloading rails in this manner is performed by six men, who can move three hundred rails per day. By simple hand-labour, formerly, twelve men were required to do the same quantity of work. The apparatus can be employed with great economy for renewals of the way. For this purpose a train of at least four trucks fitted with the apparatus is made up, and the work of lifting the worn rails and replacing them with new ones is done in the intervals between the trains.

D. K. C.

Efficiency of a Hand-Brake. By A. Pulin.

(Revue générale des Chemins de fer, February, 1890, p. 53.)

The hand-brake on the tender of a locomotive on the Northern Railway of France was tested with a view to determining the internal frictional resistance of the brake, and to ascertain the efficiency or proportion of the power applied to the handle to the pressure applied to the wheels. The brake-blocks were removed from one pair of wheels, and a conical spring was substituted for each block. The elastic strength of these springs was previously ascertained exactly. A pulley having the same radius as that of the handle on the hand-wheel was fastened on the upper end of the brake shaft, and a cord was passed round it, supporting a scale to carry weights. The scale-load was varied from 20 to 160 kilograms, or from 44 lbs. to 352 lbs., and the corresponding joint-pressures on the springs varied from 2,100 to 5,850 kilograms, or from 4,620 lbs. to 12,870 lbs.

Now, the diameter of the pulley on the screw-shaft was 11·81 inches, the pitch of the screw was 0·797 inch, the arms of the lever transmitting the force from the screw to the springs were 18·90 inches and 4·17 inches. Consequently the ratio of the force applied to the wheels to that applied to the pulley, neglecting friction, was as

$$\frac{11{\cdot}81 \times 3{\cdot}14 \times 18{\cdot}90}{0{\cdot}797 \times 4{\cdot}17} = 210 \text{ to } 1.$$

After making some slight allowances, the ratio is taken as 208 to 1, exclusive of friction, and the maximum pressure at the pulley as 158·8 kilograms, or 349 lbs. The calculated pressure on the wheels then becomes (349 × 208 =) 72,592 lbs., but the pressure actually recorded was only 12,870 lbs., or 17·7 per cent. of what was due irrespective of friction, showing that the frictional resistance absorbed 82·3 per cent. of the whole power applied. Nearly the whole of this unexpectedly large resistance is explained by adopting the coefficient of friction 0·15, and calculating the frictional resistance on that basis.

The Author concludes that in general the efficiency of hand-brakes does not exceed 20 per cent. Nevertheless, tender-wheels can be skidded by means of such brakes. Inequalities of surface are favourable for gripping. Continuous brakes, in which screw-action is dispensed with, are, no doubt, much more efficient.

D. K. C.

Compensating Gear for Rope-Sheaves. By N. J. Raffard.

(La Lumière Électrique, vol. xxxvi. 1890, p. 7.)

The Author draws attention to the fact that rope-gear, especially when used for transmitting power from large to small sheaves, is frequently unsatisfactory. When gear of this type is put down with extreme care, it may work well for years; but if the diameter of the ropes should vary, or if there should be differences in the shape and diameter of the individual grooves on one pulley, the work is not evenly divided between the different ropes, and the consequence is great wear and breakage. The Author examines three cases: 1, when the driving and driven pulleys are of the same diameter; 2, when the driving pulley is smaller than the driven pulley; and 3, when the driving pulley is larger than the driven pulley.

In the first case, absolute equality of diameter in the different ropes is not required, and a worn rope may be replaced by a new one which will ride higher in the groove than the other ropes without altering the even distribution of strains between all the ropes provided that the diameter and shape of corresponding grooves on the two pulleys are the same. Slight differences in shape and diameter of the grooves on one pulley will generally occur; but as long as such differences do not cause more than ½ per cent. variation in the linear speed of the ropes, they will be compensated by the elasticity and slip of the ropes, which the Author estimates at 1 per cent. of the linear speed.

In the second case, the ropes will work even better, the tendency being for the stouter ropes to do more than their share of the work, since they, riding higher in the grooves of the small driving pulley, receive a greater linear speed than the thinner ropes, and thus relieve the latter to a certain extent of their work. The practical effect of this is that all the ropes wear equally, and are under equal strains. If, however, a worn rope is replaced by a new one considerably stouter than the other ropes, this new rope must do very much more than its fair share of the work, and is therefore liable to be overstrained and broken.

The third case presents the greatest difficulties. There also the elasticity and slip of the ropes produce a certain compensating effect between them; but this is generally insufficient to insure good working for any length of time, the reason being, that when the larger of the two pulleys is the driver, there is a tendency to increase instead of diminish, any inequality in the diameter of the ropes. Suppose, for instance, that one particular rope works harder than the others. Its diameter will decrease, and it will ride deeper in the grooves of both pulleys. The speed ratio between the two pulleys, as determined by this rope, will then be increased, and the thinnest rope will be called upon to do most work. On the other hand, if a new and stouter rope be put on in exchange for one of the worn ropes, the speed ratio between the two pulleys, as determined by the new rope, will be less than that determined by the old ropes, and the former will take less than its share of the work or will do no work at all. The difficulties pointed out above increase with the speed ratio diameter of rope as compared with the diameter of pulleys and number of ropes. To avoid them, the Author has devised a compensating-gear similar to that used between the road wheels of traction engines, the gear being used both on the driving and driven pulleys. A diagram is given, showing the application of the compensating-gear to a rope transmission, in which four separate ropes are used.

G. K.

On Forced Draught. By — Perret.

(Compte Rendu de la Société des Ingénieurs Civil, March 1890, p. 114.)

With reference to the communications of Mr. Dumoulin and Mr. Jouffray on forced draught in furnaces, Mr. Perret holds that it is of great importance, whether for the purpose of burning coal slack, or for generating a greater quantity of steam. With rich coal as fuel, combustion with forced draught is easy, but for the slack of dry coal, non-caking, the problem is more difficult. It is necessary to reduce the width of air-space between the fire-bars to 3 millimetres (0·118 inch), or less than $\frac{1}{8}$ inch; and seeing that this width of air-space is indispensable, the least alteration of the grate is productive of serious inconvenience. The lower portion of the fire-bars, which were made sufficiently deep, was therefore immersed in water, to keep them cool and rigid, and the given width apart being thus maintained unaltered, dry coal slack could be burned continuously. The water, evaporating, passed through the burning mass and was decomposed, with marked advantage. The combustible gases became proportionally less diluted, in virtue of the relative reduction of the volume of nitrogen, and yielded a greater quantity of heat per unit of volume. This fact is evident with gazogenes, though less obvious for the fire-grates of steam-boilers.

Mr. Perret is surprised by the fact that the more the draught is forced the less is the evaporative efficiency of the fuel.[1] For instance, in a factory where the rate of evaporation was from 7 lbs. to 8 lbs. of water per lb. of coal, it fell to 4 lbs. after forced draught was introduced. Though there was a length of 400 feet of flue, and a chimney 130 feet high, a volume of flame was produced at the top of the chimney when the forced draught was in action—evidence of the production of a considerable quantity of carbonic oxide leaving the furnace unconsumed. The same phenomenon has been observed on torpedo-boats.

D. K. C.

The Metallurgical Treatment of Cupriferous Pyrites in the district of Huelva. By L. DE LAUNY.

(Annales des Mines, 1889, vol. xvi., p. 427.)

The Author, after describing the geology and structure of the great pyrites mines of San Domingos, Tharsis, Rio Tinto, and others, proceeds to consider the methods adopted for exhausting the copper. These are classified as follow:—

1. Natural cementation.
2. Artificial cementation.
3. Artificial cementation with chlorides.

The first of these methods, which is essentially one of atmospheric oxidation, is practised at San Domingos in Portugal, where the burning of pyrites in open heaps is forbidden by law. The ore, a cupriferous iron pyrites containing from 2·5 to 3·0 per cent. of copper, is broken into lumps of the size of the fist, and spread out on floors in heaps varying from 15 to 40 feet in thickness. Deep trenches are cut through the heap at regular intervals for the escape of drainage water, and a series of holes lined with earthenware pipes are sunk into it at the top, for the purpose of inserting thermometers when it is desired to ascertain the temperature in the interior. Water is distributed over the surface by a rectangular network of pipes with jets, and the operation is continued for several years, about 3,000,000 tons being under treatment at one time. The sulphates formed by the joint action of air and water dissolve in the latter, and are collected in channels leading to the cementation tanks. The surface of the heap is broken up by a pick from time to time to prevent the crust of sulphates from becoming compact and impermeable to water. The operation, though extremely simple in principle, requires somewhat delicate management, especially in regard to temperature. If the heap is too cold the sulphide of copper is not attacked, while if too hot, an excessive production of steam prevents the access of air and the consequent oxidation of copper, while a larger proportion of iron is dissolved. The best results are obtained at 100° Fahrenheit, a heat that is maintained by varying the amount of water. It is also essential that the solution shall be as rich in copper sulphate as possible, and if the liquors are too poor, the watering of the heap is sometimes suspended from fifteen days to a month.

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[1] This fact is well known and understood in steam-boiler experience, t not to the extent exemplified in the text.—D. K. C.

When the proportion of copper in the heap is reduced from 2·8 to 0·25 per cent., the heap is broken up and exported for use as a sulphur ore, the process having the advantage of saving the bulk of the sulphur. The quantity of water required being very large, two storage reservoirs have been built for holding it, one being of 5,000,000 and the other of 1,500,000 tons capacity, the latter serving also for irrigation and domestic supply.

The liquors obtained from watering the heaps are led into the cementation tanks, which are asphalted basins 13 feet square, arranged in ten series of twenty-four. These are filled with pig-iron piled chequerwise, and the coppery liquors are run through them in such a manner as to obtain the most complete precipitation with a miminum consumption of pig-iron. The precipitated copper or *cascara* deposits on the surface of the pigs, and is cleaned off at intervals of a month. The deposit, consisting of a mixture of copper with graphite, and iron rust, containing about 65 per cent. of copper, is dried upon a heated surface and packed in bags of 100 lbs. for export.

The consumption of pig-iron, which theoretically should be only 0·83, is actually 2·0 per unit of copper obtained. This is due to the formation of ferric sulphate in the liquors, which must first be reduced to ferrous sulphate by the iron, before any copper can be separated.

The exhausted liquor, essentially a ferrous sulphate solution, is stored in tanks, which, to avoid permanent poisoning of the river, are only emptied during the period of spring floods.

Artificial cementation.—This term is applied by the Author to the method of burning the pyrites in heaps as a preliminary to extraction by water and precipitation, which is in general use in Spain, although at present attempts are being made to abolish it on account of the damage done by the sulphur gases. At Lagunazo, near Tharsis, where the ore contains from 2 to 3 per cent. of copper, with about 3 per cent. of arsenic, 2·3 oz. of silver, and about 1½ dwt. of gold per ton, it is burnt in heaps called *teleras*, in a similar manner to that subsequently described as in use at Rio Tinto, for about two months.

The solution of the copper is effected in two stages, it having been found that ferric sulphate liquors will remove from one-eighth to one-fifth of the copper in the first ten days. The burnt ore is subjected to a first washing in tanks for twelve hours, followed by a second of twenty-four hours, and a third of forty-eight hours, after which it is piled into heaps called *terreros*, where the residual copper is extracted by watering, an operation which, like that in the natural cementation process, lasts for several years. The cementation is effected in tanks, through which a very rapid current is maintained, the iron being turned over every ten days; the amount of iron consumed is 1·7 per unit of copper obtained. The *cascara* when dried and roasted in a reverberatory furnace, an operation which requires about 4 per cent. of its weight of coal, is packed in bags for export.

The deposit at Lagunazo was only opened in 1882. It now produces about 120,000 tons of pyrites annually. About 1,000 tons of copper are obtained from 80,000 tons of burnt ore under treatment.

3. *Artificial cementation with chlorides.*—In this method, which is practised at Rio Tinto, the copper is dissolved not as sulphate, but as chloride. This has the advantage of reducing the waste of iron in the precipitation tanks, by avoiding the formation of ferrous and ferric sulphates, as the solution of the sulphide of copper is effected by means of ferric chloride, while iron pyrites is but slightly acted on. By this means a ton of copper precipitate may be obtained with an expenditure of an equal weight of pig-iron.

The produce of the mine is divided into five different classes, each having its own method of treatment. These are:—

1. Smalls, obtained by sifting the larger sizes. These are subjected to a modification of Doetsch's process of washing by ferric chloride, as they pack too close to be converted on sulphates by atmospheric oxidation.
2. Ordinary ores with less than 2·75 of copper. These are burnt, watered, lixiviated, and precipitated.
3. Ores of 3·75 and 4 per cent. average, obtained when necessary by the mixture of other kinds. These are sent by rail to the pier at Huelva and shipped, those of 3·75 per cent. going to England, and those of 4 per cent. to Germany.
4. Rich ores above 6 per cent. These are unfit for calcination, and are therefore smelted for regulus in water-jacket cupolas.
5. Minerals containing copper and lead. These are put aside as giving trouble in the ordinary working, and have not as yet been utilized.

Treatment of the smalls by the modified Doetsch process.—This, which like the method of natural cementation at San Domingos, does away with roasting, but reduces the loss of time and iron by adding salt, is applied in practice with the following modifications:—

On the side of a hill below the *teleras*, or roasting heaps, there are five series of tanks, each series consisting of seven tanks 15 metres long, 4 metres wide, and 1¼ metre deep, which are divided into eight compartments by walls of perforated bricks, placed edgeways and covered by planks. At a lower level is placed another series of four larger tanks, 100 metres long, 2 metres broad, and 1¼ metre deep. The whole are used indifferently for the treatment of the smalls, and for the first washing of the burnt ores, which extracts about one-fifth of their contents of copper, before sending them to the *terreros* at Naya. The raw smalls are mixed with a proportion of burnt ore, in lumps of the size of a walnut, and washed with liquors brought from the large *terreros* containing a certain quantity of common salt. This operation lasts about eight days; the wash-water going to the precipitation tanks, while the partially-exhausted mineral is taken to the *terreros* at Naya. The reactions in this method appear to be as follows:—

The burnt ore having been subjected, at a high temperature, to the action of oxygen and chloride of sodium, gives rise to—

$$2\ FeOSO_3 + 2\ NaCl + 3\ O = Fe_2O_3 + 2\ NaOSO_3 + 2\ Cl$$
$$Fe_2O_3.3\ SO_3 + 3\ NaCl + 3\ O = Fe_2O_3 + 3\ NaOSO_3 + 3\ Cl$$

The chlorine liberated, acting on the sulphides of iron and copper, produces ferric and cupric chlorides. There are therefore present $FeOSO_3$, $Fe_2O_3.3\ SO_3$, $CuOSO_3$, $Fe_2\ Cl_3$ and $CuCl$, with an excess of salt which changes the ferric sulphate into chloride. The copper in the ore, existing partly as cuprous sulphide Cu_2S, and partly as cupric sulphide CuS, the reactions become—

$$CuS + Fe_2Cl_3 = 2\ FeCl + CuCl + S$$
$$Cu_2S + Fe_2Cl_3 = 2\ FeCl + Cu_2Cl + S$$

the ferric chloride attacking and dissolving the copper, while the iron pyrites is almost untouched. The final separation of the copper is effected by cementation in the manner previously described.

Treatment of ores with less than 2·75 per cent. of copper.—This, the method of artificial cementation with chlorides, is applied to the largest part of the output of the mine. It includes the four operations of roasting, washing, precipitation, and cleaning of the precipitate, or *cascara*.

The roasting is performed in *teleras*, which are heaps of a truncated pyramidal form, 3 metres high, on a base of 6 by 8 metres for those of 800 tons, and 9 by 8 metres when containing 1,200 tons. At the bottom, one transverse and three longitudinal flues, about 20 inches square, are formed by the larger blocks of mineral. These are the firing passages, and communicate with vertical chimneys, of which there are two in the 800-ton, and three in the 1,200-ton heaps. The mass of the heap is made up of lumps a little above nut-size, salt being added in the proportion of 14 tons to 800 tons of pyrites. When the fire has been started with a little wood, it is kept up by the heat of the burning sulphur. It is essential that no rich ore shall be included, as, on account of the action of "kernel roasting," lumps of rich sulphide of copper are formed, which cannot be subsequently dissolved; and as the mass of material is so large that hand-picking is not easy, great loss of copper is likely to ensue. It need scarcely be said that the operation causes an enormous nuisance, as, since the introduction of the use of salt, the volatile products contain chlorine as well as sulphurous acid, and, especially during the prevalence of easterly winds, the village of Rio Tinto is almost uninhabitable from the sulphur-smoke, while all vegetation is destroyed to a distance of 6 leagues around. This is now sought to be mitigated by removing the roasting heaps to a greater distance—at Planes, where large lead chambers are under construction for the purpose of saving from the sulphur-gases the sulphuric-acid required for cleaning the copper precipitate, and which has hitherto been purchased abroad. A more important application has, however, been found in using sulphurous acid gas for promoting the formation of sulphates in the ore, which is now carried out on a very extensive scale at Naya.

In this method, the *teleras* made in the ordinary way, as soon as they begin to give off sulphurous acid, are covered up with a fresh quantity of mineral, partly raw and partly calcined, to which 2 or 3 per cent. of salt, and a similar proportion of peroxide of manganese have been added. The whole is formed into a heap 8 inches high with a flat top, which is divided by gutters into squares of 8 metres in the side. The remaining operations are effected in the usual way, the *teleras* being watered at intervals for months and years, the copper being slowly dissolved, and collected at the bottom of the heap. It is necessary to break up the surface with a pick at intervals, to prevent its becoming impermeable to water.

The sulphurous acid gas in this modification of the process, in the presence of steam formed by the heat developed in the heap, produces sulphuric acid, which acts upon the oxides in the crust of the roasted minerals. The salt and manganese ore may, jointly with the sulphuric acid, evolve chlorine, forming ferric chloride, which decomposes the sulphides of copper and silver. It is also possible that, under the action of heat and sulphuric acid, oxygen is evolved, which acts directly upon the pyrites.

The precipitation of the copper is effected at La Cerda, near Rio Tinto, in a series of tanks 100 metres long and 30 metres broad, divided into ten parallel series, receiving a uniform supply. The total length is about 400 metres, with a difference of level of 4 metres, giving a sufficiently rapid flow. Pig-iron as run from the furnace and scrap are used, the latter being put into baskets. Every ten days the iron is removed, and, after scraping to collect the deposited copper, is returned to the tanks. The consumption is reduced to a little more than 1 ton of pig-iron per ton of precipitate produced.

The *cascara* precipitate as collected is very impure, containing only 65 to 70 per cent. of copper, the remainder being ferric oxide, more or less arsenical, graphite from the pig-iron, silica, &c. After treatment with water acidulated with sulphuric acid, which dissolves the basic ferric arseniates without touching the copper, it is passed over sieves of $\frac{4}{10}$ inch mesh to separate any adherent masses of cast iron, which are hand-picked, and any copper separated is subjected to another washing with acid. The *cascara* passing through the sieves is washed in a current of water, where it separates the order of density into *cascara*, or copper; *grafita*, or particles of coal and graphite; and *pucha*, a fine black sand. The *cascara* is subjected to a fresh washing upon a fine wire-gauze screen, which separates a further quantity of graphite, and this passes to a series of plunger jigging machines, where it is finally cleaned. The graphite is collected and returned to the rich ore smelting furnaces.

The *pucha* is washed upon convex round buddles, 15 feet in diameter, by which a certain quantity of fine copper is saved. The waste is made into balls, which are dried in heaps and also sent to the smelting furnaces.

The silver in the pyrites usually remains in the *terreros*, a small proportion only passing into the precipitate copper.

The rich ores containing more than 4 per cent. of copper are, after burning, smelted in cupolas, or water-jacket furnaces of an American pattern, having a large exterior fore hearth or receiver in which the separation of slag and regulars takes place. The charge consists of burnt ores, old slags, a certain proportion of raw pyrites, quartz, and graphite. The regulus, which is exported to England, contains from 38 to 40 per cent. of copper; the slags with 0·4 per cent. are thrown away.

H. B.

Grabau's Method of Making Aluminium. By — Jehon.

(Annales des Mines, vol. xvi., 1889, p. 534.)

This process is based upon the reduction by sodium, of fluoride of aluminium, produced from the action of sulphate of alumina upon fluor spar and cryolite; but the latter mineral is only employed at the commencement of the operation, it being reproduced in large quantity in an artificial form, as a consequence of the reduction of the fluoride of aluminium, and of a much higher degree of purity than the natural mineral, which always contains spathic iron ore and quartz.

Production of fluoride of aluminium.—From 10 to 13 parts of sulphate of alumina, dissolved in water, is mixed with finely-divided fluor spar, and heated to 60° Centigrade for several hours, when a partial decomposition of the fluor spar takes place, giving sulphate of lime and aluminium fluoride. By repeating the operation several times, about 66 per cent. of the sulphuric acid in the sulphate may be replaced by fluoride. It is more convenient, however, not to push the change beyond 55 per cent. The result is a solution of fluo-sulphate of alumina, $Al_2 Fl_4 SO_4$, which is filtered, freed from iron by prussiate of potash, and boiled down to the consistency of syrup. This is then mixed with finely-ground cryolite to a stiff paste, giving when dried in a lead basin at 150° Centigrade a spongy mass, which is broken into pieces of the size of a walnut, and subjected to a dull red heat in a cast-iron vessel in a muffle. This decomposes the remaining sulphate of alumina, giving as a result pure fluoride of aluminium and sulphate of soda. The latter salt is washed out with boiling water, about 15 per cent. of the former also going into solution. The residue, or 85 per cent. of the fluoride in the material treated, is pressed into cakes, dried, and broken up.

Reduction of fluoride of aluminium.—The reduction of the fluoride by sodium is performed in a cast-iron vessel whose diameter is equal to its height, lined with cryolite, either rammed, or preferably in the form of bricks, made coherent with a solution of common salt.

The fluoride is heated to redness in an iron cylinder with a refractory lining free from iron and silicon, and having a cover at the top and a counterpoised drop bottom. The fluoride does not melt, and is but slightly volatile if kept well covered. The heated charge is dropped into the reducing pot, and immediately afterwards an ingot of sodium, heated nearly to its melting point, is added, the whole being covered up by an asbestos cloth. The reaction is very violent; the charge boils, and often flame coloured by sodium escapes from beneath the cover. When the proportions of sodium and aluminium fluoride are so chosen that only one-half of the latter is reduced, the remainder combines with the fluoride of sodium formed in the reduction and produces cryolite, which at the end of the operation is found as a well-melted mass, the temperature having risen to a red-white heat, having below it a lump of aluminium, covered with a thin adherent crust of cryolite. The cryolite so produced is much purer than the natural mineral, being perfectly free from iron and silicon, and in consequence the aluminium obtained is often very pure, assaying up to 99·77 per cent., according to the results obtained at the École des Mines, Paris.

The sodium used is obtained by a new method, which is only described in general terms, some details not being completely protected. It consists essentially in electrolyzing melted chloride of sodium in a crucible. One electrode is of carbon, and the other an iron wire. The latter plunges into the centre of the crucible, and is covered by a bell of porcelain with hollow sides, and a central tubulure connected with the sodium condenser by an iron tube, which carries away the globules of sodium as they form and rise to the surface; the chlorine goes to the carbon electrode.

The production of cryolite in this process is rather larger than the amount necessary for reduction, and therefore some surplus will remain for disposal. This may be used by glass-makers. As compared with Deville's process, it is said to utilize the sodium more perfectly; from 83 to 90 per cent. of the reducing effect being realized, as compared with 76 per cent.

H. B.

The Héroult Process for Manufacturing Aluminium.

By C. Talansier.

(Le Génie Civil, May 1890, vol. xvii., p. 1.)

The Author gives an historical sketch of the manufacture of aluminium, describing the method of St. Claire Deville; the cryolite process as used at Wallsend; the Cowles process, and the Minet process.

The method of Héroult consists in decomposing melted alumina by a powerful electric current between carbon electrodes. The oxygen is disengaged at the anode, burning off in carbonic oxide. The apparatus consists of a cast-iron vessel, whose sides and bottom have a thick lining of solid carbon. It is insulated from its supports. If it is desired to produce pure aluminium, a quantity of liquid cryolite is poured into the vessel, and also alumina. The negative electrode is a carbon rod, which is lowered into the bath. The metal is tapped every twenty-four hours, and alumina added as required from time to time. Aluminium bronze and iron are also manufactured in a similar way, the only difference being that melted iron or copper is poured into the bottom of the bath, and no cryolite is used. The alloys made have from 8 to 30 per cent. of aluminium, and the yield of aluminium is about ½ oz. per HP. per hour.

The process is in use at Neuhausen (on a fall of the Rhine), and in France at Froges (Isère). At the latter factory the motive-power is derived from two turbines of 300 HP. each, which drive four dynamos, each producing a current of 6,000 amperes at 15 volts, and from a smaller turbine of 100 HP., which drives an exciting dynamo, of 500 amperes and 65 volts. The small dynamo also serves to light the factory. The carbon electrodes are made at the works.

The article concludes by a description of the properties and applications of the metal, and gives a series of tests of various alloys carried out at Zurich and at Froges. Two engravings of the apparatus are given.

C. F. F.

Bibliography of the Salt Industry. By A. Aigner.

(Berg- und Hüttenmännisches Jahrbuch der k. k. Bergakademien, 1889, p. 268.)

In a monograph on the salt industry, covering one hundred pages, the Author gives a complete bibliography of the subject, classed under the following heads:—1. Treatises on heat; 2. General treatises on salt and salt-mining; 4. Methods of extracting salt—(*a*) by heating from below; (*b*) by heating from above; (*c*) by the Piccard process; (*d*) by chemical processes. 5. Appliances used in the extraction of salt—(*a*) relating to combustion (grates, chimneys); (*b*) relating to firing (smoke-consuming appliances and gas-firing); (*c*) relating to evaporation and drying. The various Papers are arranged chronologically, and abstracts are given of the more important ones. The Author then gives a detailed description of the salt-mines and works of Germany, Austria-Hungary, Holland, Great Britain, America, and Russia, and concludes with a comparative Table of the principal results deduced from these descriptions.

B. H. B.

Prince Hohenlohe's Zinc-works in Silesia. By P. Schmieder.

(Berg- und Hüttenmännisches Jahrbuch der k. k. Bergakademien, 1889, p. 389.)

The ore for the Hohenlohe group of zinc-works is obtained from the Neue Helene mine at Scharley, and the coal from a colliery belonging to the works. The smelting-works are three in number: the Hohenlohe works, with seventy-two furnaces; the Fanny Franz works, at Kattowitz, with twenty-six furnaces; and the Carl works at Ruda, with eight furnaces. Besides these there is a works—the Hohenlohe calcining works—specially devoted to the preparation of the zinc-blende for distillation. In the calcining works there are fourteen long double-bedded calcining furnaces of the Freiberg type, each with five working-doors on either side. Each furnace receives simultaneously three charges of 220 lbs. of blende, two on the upper and one on the lower bed of the furnace. The out-turn per furnace is 220 lbs. of roasted blende every five hours. The sulphur is not completely removed, the zinc sulphide being converted into oxide at a fairly low temperature (about 1,100° Centigrade), at the fire-bridge. The sulphur in combination with lead, iron, &c., is not taken into account, as it does not appear to influence the reduction when an out-turn of 78 to 80 per cent. of zinc is obtained. The loss in calcining amounts to 10 to 20 per cent. When satisfactorily roasted, the powdered ore is yellowish-green, the coarser material yellow, and the lumps reddish-brown. The two latter varieties are pulverized before roasting; whilst the slimes are dried on the top of the furnace. As cheap small coal is used as fuel, the material to be calcined must be covered with a layer of slag, and the coal moistened. In this way, the steam formed aids the oxidation. The furnace-gases, containing 1 per cent. of sulphurous anhydride, 3·5 per cent. of carbonic anhydride, 16 per cent. of oxygen, and the remainder nitrogen, pass through a flue, in which most of the dust is deposited; then several times up and down a condensing-tower, where they are subjected to the action of water and of milk of lime, and finally pass to a stack 328 feet in height. The deacidifying lime, which is sold as a fertiliser, is deposited in settling-tanks, and after dessication contains—

Lime	40·0
Carbonic anhydride	2·5
Sulphurous anhydride	37·5
Sulphuric anhydride	4·0
Ferric oxide and alumina	3·4
Residue	8·0

The three zinc-smelting works are similar in arrangement. The Hohenlohe works, which may be taken as type, contains seventy-two distillation-furnaces, each with thirty-two muffles, heated by gas from Boëtius producers. Every two furnaces form a double furnace, with 16 niches on either side, each containing two muffles. Besides these, two spaces are provided at either end of the double furnace, one serving for tempering the muffles, and the other for calcining the lump calamine. Most of the calamine is charged raw, as the carbonic acid is driven off long before the temperature of reduction is obtained. The charge per furnace is 507 lbs. of ore, of which 187 lbs. consists of roasted blende containing 42 to 45 per cent. of zinc, 55 lbs. of lump calamine with 26 to 32 per cent. of zinc, 33 lbs. of calamine slimes with 20 to 23 per cent. of zinc, and 232 lbs. of poorer slimes with 14 to 16 per cent. of zinc. The daily out-turn of a furnace is 108 lbs. of zinc. The zinc contains 3 to 4 per cent. of lead, which is reduced by refining to 1·07 per cent. Besides this, the refined zinc contains 0·02 per cent. of iron and 0·04 per cent. of sulphur. The refining is effected in a reverberatory furnace with an inclined base, in the well of which the lead collects at the bottom, and the zinc above it. Every twelve hours, 1,102 lbs. of zinc is tapped, and the same amount of raw zinc charged in; and the lead is removed every eight days. The zinciferous lead is melted in a pot of 4 to 5 tons capacity, and the zinc-scum removed as it is formed.

Cadmium is extracted from the zinc-dust collected in chambers under the muffle furnaces. The extraction is effected in one of the distillation-furnaces, in eight muffles with receivers, the charge being 11 lbs. of zinc-dust and 66 lbs. of cinders. Owing to the low temperature prevailing in the portion of the furnace where these muffles are placed, cadmium oxide is reduced. Cadmium-vapour enters the receivers and is there partly oxidized. The mixture of cadmium and cadmium-oxide, about 11 lbs., is heated with an equal weight of charcoal, in a retort holding 27 lbs. of the mixture. About 13 lbs. of raw cadmium is obtained, which, when remelted in an iron crucible with salt, yields about 11 lbs. of refined cadmium, containing 99·6 per cent. of cadmium, 0·04 per cent. of zinc, 0·02 per cent. of iron, and 0·15 per cent. of lead.

B. H. B.

The Przibram Smelting-Works. By A. Zdráhal.

(Berg- und Hüttenmännisches Jahrbuch der k. k. Bergakademien, 1890, p. 1.)

The metallurgical process now adopted at the Przibram works, in Bohemia, has not yet been described in its entirety. The Author, who is manager of the works, has therefore been induced to publish an exhaustive description, covering eighty-six pages, and illustrated by twelve plates drawn to scale. The statistics given in the Paper refer to the year 1888. During that year the works treated:—

—	Weight.	Content of Metal.	
	Tons.	Silver in lbs.	Lead in tons.
Ores	13765·8	80831·7	4942·9
Products from lead mill	3·2	—	2·8
Old slags from tip	1680·0	221·7	26·8
Products previously obtained	2280·5	9697·6	1100·5
Total	17729·5	90751·0	6073·0
Less final products	2421·0	11725·3	1080·8
Total treated	15308·5	79025·7	4992·2

From this there resulted a production of:—

—	Weight.	Content of Metal.	
	Tons.	Silver in lbs.	Lead in tons.
Fine silver	35·07	77159·9	..
Refined lead	1344·7	..	1344·7
Antimonial lead	321·1	..	321·2
Red litharge	1747·6	..	1590·3
Green litharge	939·1	..	854·6
Total	..	77159·9	4110·8
Loss	..	1865·8	881·4

The loss may be referred to the following processes:—

	Per cent. of Loss of Silver.	Per cent. of Loss of Lead.
Smelting and calcination	37·22	51·83
Liquation	6·55	3·73
Pattinsonising	6·09	4·24
Refining and remelting	0·01	1·13
Cupellation	34·68	33·52
Reduction of litharge	..	3·28
Smelting for hard lead	0·09	2·27
Finished litharge	12·67	..
Finished lead	2·69	..
	100·00	100·00

The total loss is 2·308 per cent. of the silver contained in the ores, and 17·822 per cent. of the lead. The total cost of treatment is as follows:—

	£	Per cent.
Pulverising	1,366	3·48
Roasting in furnaces	8,162	20·79
Roasting in stalls	280	0·71
Smelting	19,927	50·77
Liquation	600	1·53
Pattinsonising	2,367	6·03
Refining	373	0·95
Cupellation	4,727	12·04
Reduction of litharge	458	1·17
Smelting antimonial lead	992	2·53
	39,252	100·00

The metallurgical process employed at Przibram may be summarized as follows:—

I. *Roasting in furnaces.*—The furnaces used are long-bedded calciners, of which there are twelve. The materials treated are: All the ores, the flue dust from roasting (2), flue dust from smelting (11), flue dust from Pattinsonising (20), flue dust from cupellation (36), furnace dust (10), and rich marl (40). The products are: 1. Roasted ore, which passes to the ore-smelting, III; and 2. Flue dust, which is returned to the furnace, I.

II. *Roasting in stalls.*—This is also effected in open heaps. The materials treated are: Ordinary lead regulus (7), cupriferous lead regulus (49), and furnace dross (8). The products are: 3. Roasted lead regulus (to III); 4. Roasted cupriferous regulus (to XII); and 5. Roasted furnace dross (to III).

III. *Ore-smelting.*—This is effected in blast-furnaces, of which there are eight. One is rectangular in horizontal section, and has four tuyeres; the others are circular, and have eight tuyeres. The materials treated are: Roasted ore (1), roasted lead regulus (3), roasted furnace dross (5), poor dross (9), lead-pot scum (16), scum from Pattinsonising (19), ordinary scum from refining (22), litharge from refining (24), scum from remelting (26), scum from cupellation (28), light litharge from cupellation (32), hearth from cupellation (33), hearth dross from cupellation (34), gutter-hearth (35), test from silver refining (38), hearth from silver refining (39), and rich ore slag (12). The products are: 6. Argentiferous lead (to IV); 7. Ordinary lead regulus (to II); 8. Furnace dross (to II); 9. Poor dross (to III); 10. Furnace dust (to I); 11. Flue dust (to I); 12. Rich slag (to III); and 13. Poor slag (thrown away).

IV. *Liquation.*—This is effected in a reverberatory furnace. The materials treated are: Argentiferous lead from ore-smelting (6), and argentiferous lead from smelting the refractory dross of the liquation process (48). The products are: 14. Liquated argentiferous lead (to VIII and to V); 15. Refractory dross (to XII) and 16. Lead-pot scum (to III).

V. *Pattinsonising.*—This is effected with the aid of steam by the Rozan modification of the Pattinson process. The pots hold seven tons of lead. The materials treated are: Argentiferous lead (14), and argentiferous lead from the reduction of litharge (41). The products are: 17. Rich lead (to VIII); 18. Poor lead (to VI); 19. Scum (to III); and 20. Flue dust (to I).

VI. *Lead refining.*—The poor lead (18) from the Pattinson process is treated; the products being: 21. Refined desilverised lead (to VII); 22. Scum (to III); 23. Tin-bearing scum; and 24. Litharge (to III).

VII. *Re-melting the refined lead.*—The refined desilverised lead (21) is melted in two four-ton cast-iron pots. The products are: 25. Soft lead, which is sold; and 26. Scum (to III).

VIII. *Cupellation.*—A portion of the liquated argentiferous lead (14), and all the rich lead (17) from the Pattinson process, is cupelled. For this purpose four old, small, circular cupellation furnaces, and six large rectangular furnaces, are provided. The small furnaces are rarely used. The products are: 27. Silver (to IX); 28. Scum (to III); 29. Black litharge *a.* (to X); 30. Black litharge *b.* (to III); 31. Poor litharge, red and green (sold); 32. Rich litharge (to III); 33. Hearth (to III); 34. Hearth scrap (to III); 35. Gutter hearth (to III); and 36. Flue dust (to I).

IX. *Silver refining.*—The silver (27) from the cupellation furnace, containing 95 per cent. of silver, is refined in a reverberatory furnace. The products are: 37. Fine silver; 38. Test (to III); 39. Hearth (to III); 40. Rich marl (to I).

X. *Reduction of litharge.*—The heavy black litharge *a.* (29) from the cupellation is melted in a reducing atmosphere in a small circular cupellation furnace. The products are: 41. Reduced lead (to V); and 42. Black litharge slag (to XI).

XI. *Antimonial lead smelting.*—This is effected in the rectangular blast-furnace. The materials treated are: Black litharge slag (42), furnace dross (44), furnace dust (45), flue dust (46), and lead bottoms (47). The products are: 43. Hard lead (sold); 44. Furnace dross; 45. Furnace dust; 46. Flue dust; 47. Lead bottoms.

XII. *Refractory dross smelting.*—The refractory dross (15) from liquation consists of 70·66 per cent. of lead, 2·57 per cent. of iron, 2·05 per cent. of arsenic, 2·01 per cent. of antimony, with sulphur and oxides. The products obtained on smelting are: 48. Argentiferous lead (to IV); and 49. Cupriferous lead regulus (to II). B. H. B.

The Pollak Storage-Battery. By G. Lippmann.

(Comptes Rendus, vol. cx., 1890, p. 569.)

The Author describes a new process for the more rapid formation of storage plates than is possible by the Planté process. He uses metallic lead plates, the surface of which has been mechanically roughened so as to present the appearance of a coarse brush, the points being 1 millimetre apart, and 2 millimetres high. After washing to eliminate greasy matter from the surface, the plate is covered with a paste of sulphate of lead moistened with salt water, and placed between two zinc plates in salt water. In this way a coating of electrolytically-formed spongy lead is obtained. Plates thus prepared are made up into complete batteries, and "formed" during fifty hours, after which time, the adherence of the active material on both positive and negative plates (peroxide and spongy lead respectively), is said to be perfect. The Author gives the results of testing a cell containing four positive and five negative plates (total weight 25 lbs.), and formed during forty-five hours, with a 16-ampere current, from which it appears that a total capacity of 95·4 ampere-hours has been obtained with a drop from 2·12 to 1·8 volts. The same cell, recharged with 16 amperes during seven hours, gave, in a subsequent test, 102·35 ampere-hours discharge, or an ampere-hour efficiency of 91·38 per cent. The capacity per lb. of lead is 4·1 ampere-hours. G. K.

The Calorimetric Value of the Unit of Energy in the Electromagnetic System of Measurement. By E. Raverot.

(Lumière Électrique, vol. xxxv., 1890, p. 453.)]

The mechanical equivalent of the calorie is $4 \cdot 2 \times 10^7$ ergs, and inversely the heat equivalent of the erg is $0 \cdot 2381 \times 10^7$ calories (*sic*). The specific heat of air according to Regnault is 0·2375, and according to Wiedemann 0·2389, the mean between the two figures being 0·2382. Since the Joule equals 10^7 ergs, the Author defines it as the energy which corresponds to the variations of 1° Centigrade temperature of 1 gram of air at atmospheric pressure, and since this variation of temperature is accompanied by a variation of $\frac{1}{273}$ of volume he gives the following definition:—The Joule is the energy represented by the expansion of 1 gram of air at atmospheric pressure by $\frac{1}{273}$ of its volume.[1] G. K.

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[1] See also Blakesley "On some facts connected with the systems of scientific units of measurement." Phil. Mag., vol. xxvii., 1889, p. 178.

The Walker Electrical Meter.

(Electrical Engineer, New York, 1890, p. 207, 3 Figs.)

This apparatus embodies several novel features, and places at the disposal of the consumer a complete record of the operation of his circuit and the current consumed. The current-strength is, so to speak, continuously photographed. The meter consists of an ammeter coil, composed of a solenoid in which an iron wire, in the shape of a nearly complete circle, is placed as a core. On the same shaft to which the circle or core of the ammeter is attached there is fixed a light aluminium sector, provided with a vertical slot; and immediately behind this sector is a box, which encloses a roll of sensitized photographic paper; a screen is placed between the sector and the paper, provided with a horizontal slot, over which the sector moves; a horizontal row of holes is also made in the screen; an electric lamp is placed in a suitable position, and a ray of light can then pass through the vertical slot in the sector, and the light passing through the row of holes in the screen records a series of lines on the travelling band of paper corresponding to different strengths of current. A clockwork movement, which is wound every hour by a small electric motor, causes the paper band to travel, and also moves a shutter every hour, which allows the light to record a cross set of lines on the paper for registering the hours. At intervals, say of a month, the paper can be taken out, developed, and measured up, just as an indicator-card would be. The current consumed is very small, $\frac{3}{16}$ of an ampere being sufficient to start the apparatus. Tests were made over the whole range of the instrument, and the load was varied by single lamps, using from 0·35 to 0·62 ampere each, these small differences being distinctly recorded. The accuracy is, no doubt, due to the absence of friction in the parts, and to the fact that the core is relieved of all mechanical work.

E. R. D.

On Voltmeters Free from Temperature-Error. By Dr. K. Kahle.

(Elektrotechnische Zeitschrift, 1890, p. 270.)

In electromagnetic voltmeters, the current passing through the solenoid and influencing the position of the pointer, depends upon the pressure at the terminals of the solenoid, and upon the resistance of the latter. If on account of heating the resistance increases, the instrument must indicate less than the actual pressure, or if the instrument has been calibrated with due regard to the influence of temperature, it can only indicate correctly if continuously in circuit at approximately constant voltage, as any change of voltage cannot immediately alter the resistance to that value which obtained at the time of calibration. To remedy this defect, the Author has devised a method for winding voltmeter solenoids so as to make their magnetizing power independent of temperature. This object is attained by winding the solenoid with two wires, which are coupled in parallel in such way as to oppose each other. The main wire magnetizes and the auxiliary wire demagnetizes the coil, but as the auxiliary wire has a larger resistance than the main wire, the magnetizing action of the latter predominates. If the main wire has a small and the auxiliary wire a large temperature coefficient, it is easy to see that, with increasing temperature, there must be a reduction in the demagnetizing power of the auxiliary wire, whilst at the same time the magnetizing power of the main wire is also reduced. In order that the resultant magnetic effect may remain the same, it is therefore necessary that the difference between the ampere turns of both coils should be the same for all temperatures, a condition which the Author shows to be fulfilled if the ratio between the resistances of the two windings is the same as that between their respective temperature coefficients.

It is obviously advantageous to employ for the main winding a metal of low resistance, and for the auxiliary winding a metal of high resistance. Since, however, the temperature coefficient of good conductors is generally rather high, and that of less good conductors rather low, the condition for compensation above stated cannot be fulfilled without some accessory appliance, and this consists in an external resistance coil of low temperature coefficient placed in series with the main winding. The main winding in the instruments, made according to the Author's plan, consists of copper, the external resistance of nickel-copper alloy, and the auxiliary winding of iron. The greatest current density permitted in the main winding is 2,250 amperes per square inch; and the final temperature corresponding to this density is said to be 50° to 60° C., which can be safely sustained by the insulating materials. The insulation resistance of the instrument is 10 megohms. The Author gives very fully the mathematical theory of his method of winding voltmeters.

G. K.

Puluj's Telethermometer.

(Sitzungsberichte der k. Akademie der Wissenschaften, Wien, 1889, p. 1502.)

The action of this instrument is based upon the fact that the electric resistance of metals increases and that of carbon decreases with rising temperature. In a glass tube of 3½ inches length and ⅛ inch diameter are enclosed two parallel conductors, one consisting of a carbon filament and the other of a spiral of iron wire. The glass tube is sealed and filled with dry hydrogen to facilitate the transfer of heat between the outside and the conductors. The latter are joined at one end and a potential or balance wire is carried from the point of juncture through the glass to the outside, whilst the two other free ends of the conductors are joined to two other wires, also brought outside through the glass. The three external wires are carried to the spot where the measuring apparatus is installed. This consists of a straight wire bridge, Leclanché cell, key, and standard resistance coils. The two outer wires leading from the glass tube are attached to the terminals of the bridge, whilst the centre or balancing wire is attached to its slider through a delicate galvanometer or a telephone, in which latter case the key is replaced by an interruptor, consisting of a steel spring with carbon contact. In making a test the slider is brought into such a position that the galvanometer shows no deflection, or the telephone is silent, and the temperature can then be read off an empirically-constructed scale. The inventor of the apparatus prefers the use of a continuous current and galvanometer in preference to an interrupted current and telephone, and states that an alternating current is inadmissible on account of the disturbing influence of self-induction.

The theory of the apparatus is given at great length, and the Author finds that to obtain accurate indications over a distance of 1,100 yards the line wires must be 150 mils diameter. It was also necessary to investigate the law according to which the resistance of carbon filaments varies with temperature. Calling R_0 the resistance of such a filament at 0°C, and R that at T°C, then the relation between temperature and resistance is expressed by the formula—

$$R = R_0 (1 - 0{\cdot}00022194\ T \times 0{\cdot}00000003374\ T^2).$$

The telethermometer may also be constructed as an inspectional instrument by replacing the bridge by a differentially-wound galvanometer, the needle of which indicates at a glance the temperature existing at the distant point on a scale empirically graduated.

G. K.

The Wirt Lightning Arrester.

(The Electrical Engineer, New York, 1890, vol. ix., p. 311.)

The ordinary form of lightning arrester adopted for telegraphic circuits is not sufficient for circuits carrying the heavy currents employed for lighting, &c., owing to the fact that following in the path of the lightning stroke an arc is maintained by the current itself. The object of the invention here described is to prevent the possibility of the maintenance of such an arc by affording for the passage of the lightning discharge a series of metallic surfaces, which would present to the arc sufficient cooling effect to cause its extinction.

The apparatus is extremely simple, and consists of a pile of metallic discs separated by very thin laminæ of mica (each about 0·002 inch thick), the number of discs being proportioned to the electromotive force of the circuit, and the total thickness of all the insulating laminæ being at the same time very small; thus, if the top disc be connected to the line, and the bottom disc to earth, a series of very minute breaks is presented to the passage of the lightning stroke, while it has been found that a current sufficient to maintain an arc 4 inches long in air, would not hold an instant across this discharger, where the aggregate length of all the breaks was under $\frac{1}{20}$ inch.

As this arrester needs neither safety fuze nor inspection, it is applicable to all classes of circuits.

F. J.

A Note on the Tensions adopted for Telegraphic and Telephonic Wires. By A. BARBARAT.

(Annales Telegraphiques, vol. xvii., 1890, p. 163.)

The Author published in 1887 the following formula for telegraphic and telephonic lines:—

$$\theta = \frac{a^2 d^2}{24 a_1 q^2}(K^2 - K_0^2) + \frac{\epsilon_1 q}{a_1}\left(\frac{1}{K_0} - \frac{1}{K}\right),$$

where ϵ_1 is the coefficient of extension per kilogramme and per metre of a wire 1 millimetre section, multiplied by 10^6;

a_1 the coefficient of linear expansion multiplied by 10^6;

d the density;

q the breaking-weight in kilograms per square millimetre section;

a the span;

$\frac{1}{K_0}$ the factor of safety at the lowest temperature;

$\frac{1}{K}$ the factor of safety at which the wire is working;

θ the temperature counted from the point where $\frac{1}{K_0}$ is the factor of safety.

The Author gives four Tables worked from these figures for various values of $\frac{1}{K}$, and for various temperatures and spans, and for iron, steel, and bronze.

The Tables are graphically exhibited in two sheets of curves. The Author gives the following values for the constants in the formula:—

—	Iron.	TelegraphicBronze.	Steel.	Telephonic Bronze.
a_1	12·35	17·5	11·0	17·5
ϵ_1	54	78	45	78
d	7·8	9·15	7·8	9·3
q	$5d. = 39^k$	$5d. = 45{\cdot}75_k$	$15d. = 120^k$	$8d. = 65^k$

Ll. B. A.

Development of Generating Stations for Power and Incandescent Light. By C. J. Field.

(Electrical World, New York, vol. xv., p. 136, 6 Figs.)

In this Paper, which was read at the Kansas City Electric Convention, the Author takes as a representative type of present development, and of first-class work, the Edison Electric Illuminating Company's station, at Brooklyn, which was completed last fall, and is now in successful operation.

The boilers and engines are located on the first floor, everything thus being directly under the eye of the chief engineer. The boilers are Babcock and Wilcox's largest type of sectional water-tube boilers. The engines are 300 HP. compound horizontal automatic engines by the Ball Engine Company.

On the second floor are twenty-four dynamos each giving 720 amperes and 140 volts, and each weighing about 8 tons. Through the centre of the dynamo-room runs the electrical gallery, from which the workings of the dynamos and other apparatus, as well as all outside lines, are controlled, and from this gallery run all the feeders which connect into the mains, the net-work of which covers an area of about 1½ square mile.

The ampere-meters are located on each feeder, showing the load in each part of the district. This plant maintains its distribution, and the regulation thereof, by balancing within itself. All circuits are underground, there being about 25 miles of underground conductors. On the second floor are also located the coal storage, water-tanks, and feed-water heater. On the top floor are the offices, supply-room, and workshops; and in the basement are the ash-pits, smoke-flues, pump-rooms, coal storage for 1,000 tons, and air-blast for forced draught. The building is 100 feet long by 75 feet wide, and contains a complete installation for generation and supply of current, and power for forty thousand lights of 16 candle-power each, and the total cost, as shown by construction accounts and estimates, amounts in round numbers to £82,500. The underground lines have a capacity for twenty thousand lights, and to complete the plant for its entire capacity would amount to about £40,000 additional.

To secure an earning basis, not less than five thousand lights must be used, with an average income of £2 per light per year to clear the general, incidental, and operating expenses. This figure may be considered as the unit of operating capacity. The ratio of variable and fixed operating expenses is constant. In such a station the fixed expenses are 75 per cent. of the whole, and the variable 25 per cent., so that if the business or income is doubled, the expenses are only increased 25 per cent. The average load for the twenty-four hours is about 25 per cent. of the maximum. The working of the engines in this station shows a total of 238·3 I.HP., with a boiler-pressure of 110 lbs., revolutions 223; and load 1,200 amperes, which is the equivalent of two thousand seven hundred and twenty-five lamps, giving for the indicated power eleven and a half lamps of 16 candle-power per HP. This is for power developed.

The friction of the engine is less than 5 per cent. of its normal power. After giving a general outline of this plant, the Author considers the points in connection with the type of engines, boilers, dynamos, underground system, &c., to be adopted in a station of this class. He discusses very fully the comparative merits of the Corliss engines, and high-speed engines, and claims for the Edison underground system, that it is the most practical solution for circuits of less than 400–500 volts potential, giving as it does unequalled flexibility, and the best local distribution, and doing away with the problem of splicing and cutting of cables, which at its best is bad work.

E. R. D.

The Electric Lighting of Railway Trains.

By E. Sartieux and L. Weissenbruck.

(Annales Telegraphiques, vol. xvii., 1890, pp. 58 *et seq.*)

The Authors give a *résumé* of the report presented to the International Railway Congress, on the subject of electric lighting of trains in 1889, dividing the subject into two heads, the technical question of how it may be and has been carried out, and the commercial question, at what expense, in comparison with gas and oil. The technical question involves a description of the various methods that have been tried under the following heads:—

1. Primary batteries; 2. Secondary batteries, or accumulators; 3. Accumulators charged by a dynamo driven by one of the carriage axles; 4. Accumulator charged by a dynamo driven by a

steam-engine on the locomotive; 5. The electric lighting of trains passing through tunnels, by means of fixed generating plant, the train making contact with a rail laid in the tunnel.

Taking these in order, the first system has been tried in England, America, and on the continent; but may be said to be now definitely abandoned.

Lighting by means of accumulators is now adopted in almost every instance, in one or other of the methods above, viz., 2, 3, or 4.

Prolonged experiments on the Pennsylvania railroad, and also on the Boston and Alleghany railroad, have shown that under working conditions, an efficiency of 60 per cent., and an output of 4·5 ampere-hours per pound of electrode, or 3 ampere-hours per pound weight total, may be obtained from cells of the E.P.S. form (lead grids pasted with lead oxides).

Experience has also shown that it is preferable to use low voltlamps—not exceeding 30 volts, and to use a lamp of 6 or 8 candle nominal, with reflectors.

Experience on the railways quoted above, showed that an annual depreciation of 5 to 10 per cent. for the negative plates, and of 30 per cent. for the positive plates, would represent a fair average. The average allowance now made in America for a depreciation fund is 25 per cent. on the original total cost of the cells. The lamps last from 800 to 1,200 hours; but more generally about the former, the vibration being the chief cause of breakage of the filaments. On the State Railways in Belgium, the system tried is the same as that used in England on the London, Brighton and South Coast Railway, consisting in the use of a dynamo driven from the axle of the guards van, in which also is placed the battery of accumulators, the connection from the dynamo to the accumulators being broken when the speed of the train falls below a certain limit, and an extension of this principle was adopted by the Southern Railway in Austria, where a centrifugal governor varied the number of accumulators, in correspondence with the velocity of the train, so as to produce a constant electromotive force at the terminals of the dynamo feeding the lamps. In another system, tried by the management of the Frankfort-on-Main line, the accumulators were in series with the lamps (the latter, however, being in parallel with one another), as long as the train was running, the cells being in two parallels. When the train stops, and the accumulator alone feeds the lamps, the two parallels are coupled in series, and disconnected from the dynamo. The system is very complicated in its details.

In some trials on the State Railway of Wurtemburg, a different arrangement was made. Each carriage had its own accumulators, and these with the lamps were fed from one pair of mains, connected to a dynamo in the guard's van. An automatic switch inserted or removed a resistance from the lamp circuit of each carriage, according as the cells were charging or discharging. The dynamo itself was controlled by an automatic regulator, and the dynamo cut off when the speed became less than about 18½ miles per hour.

Even with all these arrangements there were sensible variations in the light when they acted; so two sets of cells were arranged in each car, one of which was charged whilst the other supplied the lamps, whilst in a further extension of the same plan, all the cells being charged were connected to the dynamo mains, and all the cells feeding the lamps were connected to the lamp mains, which also went right through the train. The system used on the Midland Railway is the same as the first tried at Wurtemburg, with the addition of a third wire carried along the train, and allowing the guard to extinguish the lights when unnecessary.

The use of a separate engine driving a dynamo on the locomotive, has been tried in America, and also in England on the Great Eastern and Great Western Railways. No details are given as to results.

The only two cases at present of lighting trains in tunnels from a central rail fed by a fixed dynamo are the Glasgow City Railway, and the tunnels on the line between Kieff and Odessa, whilst on the Fichburg Railway (U.S.), the whole tunnel is itself lighted by 16 candle-power lamps.

The Authors give Tables showing the cost of the various systems, which may be summarized thus:—

Systems adopted.	Cost of Plant per Lamp of 5 Candle-power.				Working cost per Lamp of 5 Candle-power per Hour.	
	£.	s.	£.	s.	d.	d.
1. Primary batteries	No data.				1·2	
2. Accumulator alone (charging dynamo not included)	8	4	6	4	0·48	0·2
3. Accumulator and dynamo driven from axle	15	10			0·45	..
4. Accumulator and dynamo driven by separate engine	14	0			0·49	0·3

The only information given as to the cost of lighting trains in tunnels by means of a central rail is, that in Russia the cost is one-third of that of gas.

The comparative cost of lighting by gas and oil is given as follows: gas from 0·36*d.* to 0·48*d.*, and by oil, 0·54*d.* As to deadweight carried, the accumulators necessary for a saloon-carriage requiring twenty-one lamps, weigh 1,200 lbs., and the apparatus for Pintsch gas illumination for the same carriage would weigh from 990 lbs. to 1,320 lbs.

Ll. B. A.

Rheostats for Stage Lighting. By E. LÖBBECKE.

(Elektrotechnische Zeitschrift, April 1890, p. 234.)

The title of this article, correctly translated, is, "The Application of the Fundamental Physiological Law of H. and W. Weber to Stage Lighting," but the shorter title at the head of this abstract will better convey an idea of the practical scope of the Author's investigation. The law in question states that the intensity of any physiological sensation varies as the logarithm of the intensity of the cause producing such sensation. On the stage of a theatre it is frequently necessary to vary the degree of illumination, which is done by the use of rheostats; but to obtain the best scenic effects it is necessary to let the variation take place gradually, and in such way that the intensity of the sensation produced by the illumination shall be augmented or diminished by an equal amount at each change. If the sensation shall advance by equal steps, as represented by the figures 0, 1, 2, 3...., then the light intensity must vary as 1, 10, 100, 1000.... ; and to get equal differences in sensation, the ratio of succeeding light intensities must be constant. Thus, if the rheostat is constructed with steps to produce the light intensities L_1, L_2...., L_{n+1}, the ratio between the brightest light, L_1, and the dullest light, L_{n+1}, is $\frac{L_1}{L_{n+1}} = C^n$, where C is the constant ratio between any two succeeding intensities.

The Author gives, as a practical example, a rheostat with nineteen steps, by means of which the light of 16 candle-power Edison glow-lamps is to be toned down to 0·24 candle-power, so as to produce the most gradual possible change in the sensation. The following Table gives the candle-power, current, and resistance in circuit at each step :—

Candles	16	14·6	13·3	12·1	10·9	9·8	8·7	7·7	6·7	5·8
Current	1	0·978	0·962	0·945	0·927	0·905	0·888	0·868	0·847	0·824
Ohms .	0·0	1·7	3·6	5·6	8·0	10·6	13·4	16·4	20·0	24·0

Candles	5·0	4·2	3·5	2·9	2·3	1·8	1·3	0·88	0·53	0·24
Current	0·804	0·775	0·746	0·722	0·690	0·660	0·625	0·600	0·577	0·555
Ohms .	28·0	32·7	37·2	41·4	46·4	50·4	54·6	59·0	62·7	66·4

The Author points out that the law between candle-power and current passing through the lamp is not the same for all types of lamps, and is even liable to variation with different lamps of the same type. For this reason he recommends that the law be determined for each lamp separately, and that the mean values be taken when it is required to construct a rheostat for a bank of lamps.

G. K.

The New Westinghouse Alternating Arc-Lighting System.

(The Electrical Engineer, New York, 1890, p. 149, 14 Figs.)

This system, broadly speaking, consists in operating a constant current main circuit which passes through converters suitable for the number of lamps to be operated at any one point. The dynamo closely resembles the Westinghouse alternating incandescent machine, but the armature coils are not as heretofore flat and placed on the periphery, but are oblong and wound separately, and then clamped in position around the cores of the armature projections, which are provided with overlapping teeth. After the coils have been placed in position the spaces between the teeth are filled out with wooden wedges, which are dovetailed and slid in from the side, so that no further fastening is required. The peculiar construction of the armature maintains the current constant at all loads; so that no regulating apparatus is required for the purpose. An apparatus for short-circuiting the armature is provided, as a break in the external circuit causes the generation of a heavy current in the armature, which, if not prevented, would eventually cause its destruction. The machines now built vary in capacity from twenty-five to two hundred and forty lights. The speed for a two-hundred-and-forty-light machine is only 480 revolutions per minute, and the number of alternations per minute is only seven thousand five hundred instead of sixteen thousand for the incandescent machines. The converters employed instead of reducing say from 20 to 1, are in the proportions of 3 to 1, or even 1 to 1. Two types of lamps are employed, a long pattern for outdoor use, and a short pattern for indoor use; the carbons for the long lamps are $9\frac{1}{2}$ inches × 2 inches × $\frac{7}{16}$ inch, and burn for about forty hours, and for the indoor pattern are somewhat shorter; there is a slight humming noise produced when burning. A special fuse is used which avoids the danger of the contacts being burnt when the fuse gives way, and consists of a piece of German silver wire placed as a shunt to the fuse, so as to melt immediately after it and form a break of 6 inches.

The efficiency of a dynamo for one hundred and twenty lights and 30 amperes current, is with forty lights on about 63·7 per cent., and with full load 87 per cent., and the energy per lamp required is 0·75 HP. Many of these machines are now in use.

The regulation for constant current depends upon the fact that the armature is wound with a sufficient number of turns to produce a high self-induction, and, consequently, a lag in the current approaching 90° when short circuited. The current characteristic of the machine can be made to approach as nearly to constancy as can the electromotive force characteristic of a constant potential dynamo with varying current. A Table of power and efficiency tests is added.

E. R. D.

Electrical Tramway Data. By C. T. Hutchinson.

(The Electrical Engineer, New York, 1890, p. 239.)

The Author gives in this Paper an account of measurements made upon the electric tram-roads of the Sprague Company. All the roads are of the overhead underrunning trolley-system, with main conductors parallel to the trolley-wire along the side-walk, connection being made at short intervals. The roads are referred to as A, B, C, D, and E.

Road A is divided into two parts by an incline which is worked by cable. The lower part of the line has about 1·03 mile, and the upper 1·92 mile of double track. The rails are used as return. The road is very uneven, one grade being as high as 15 per cent.

Power.—The power-house is at the middle of the line, and contains two tubular boilers rated at 90 HP., 90 lbs. pressure, which supply steam for the cable-engine and for the engine used for the dynamos.

Two engines, the first a Ball, 16 inches by 16 inches, 150 HP., 240 revolutions; the second, a Greenwald (Brown), 18 inches by 42 inches, 150 HP., 70 revolutions. Each drives two 50-kilowatt Edison shunt dynamos, 500 volts, at 845 revolutions. Usually twenty cars are run, but only ten were in operation when tested. Two tests each, occupying 8½ hours, were made. The average number of passengers per trip was not more than six or seven, the insulation resistance 25,000 megohms; for the entire system, about 6 miles of pole, this is 0·15 megohm mile.

Road B is almost level and divided into two branches, which run together for about 2·5 miles and then diverge; on one route there is about 5·5 miles, and on the other about 4·94 miles of double track.

Boilers.—Seven tubular boilers 18 feet by 6 feet, 150 HP., 90 lbs. pressure.

Engines.—Armington and Sims, 18½ inches by 18 inches, 200 revolutions, 250 HP.; three Armington and Sims, 14½ inches by 15 inches, 260 revolutions, 125 HP.; each engine is belted direct to two dynamos.

Dynamos.—Six 80-kilowatt compound wound, six 40-kilowatt shunt wound.

The station runs twenty-four hours a day. The average number of cars upon the line is thirty. The insulation resistance was about 6·9 megohm miles for about 15 miles of poles.

Road C.—This road is about 4 miles long and is entirely single track, with turnouts; there are many grades, some of 7 or 8 per cent. The road is in the form of an L with the power-house at the junction; one branch has a return copper conductor, the other uses the rails connected at intervals to buried earth-plates for the return.

Boilers.—Two tubular, 18 feet by 6 feet, rated at 150 HP.

Engines.—Three Ball, 16 inches by 16 inches, 240 revolutions, 150 HP., belted direct to dynamos.

Dynamos.—Six 50-kilowatt Edison.

Nine or ten cars run constantly. Round trip about 8 miles. The load in all cases was about 9,000 lbs. to 10,000 lbs., and the speed from 7 miles per hour to 5 miles per hour on grades. Insulation resistance, 3·2 megohm miles, with about 8 miles of poles.

Road D.—The road is quite level. There is a main line with six branches tapping the main line. The power-house is at one end of the line.

Boilers.—Two tubular, 16 feet by 6 feet 6 inches.

Engines.—Two Ball, 16 inches by 16 inches, 250 revolutions, 150 HP.

Dynamos.—Four 50-kilowatt Edison, shunt wound.

About seventeen cars are equipped, and fourteen of them run for eighteen hours. The speed was about 6 miles per hour. Insulation about 4 megohm miles.

Road E.—There are no grades on the main line of this road, it has about 2·3 miles of double track. Eight motor cars run on an average; but for an hour in the afternoon ten motor-cars run, six of them with tow cars.

Boilers.—Two Babcock and Wilcox, 150 HP., boilers.

Engines.—Two 18 inches by 42 inches Corliss condensing engines of 150 HP.

Dynamos.—Two 80-kilowatt shunt-wound Edison dynamos.

Neither ground wire nor earth-plates are used; the return is made entirely by the rails connected to each other by copper bonds.

The insulation was 0·5 megohm miles for 8 miles of poles. Table I is compiled from the results obtained on all the five roads, Tables II and III from the results obtained on A, B, D, and E.

The figures in column 4, Table I, are obtained by deducting the values on the friction-card from the mean I.HP. and dividing by the number of cars. The car readings were taken at 30-second intervals; but it is found that this does not give a good result, and that the only fair plan is to have a recording meter which will give a continuous record. These roads were not selected as being the best, but were chosen simply as being conveniently situated.

TABLE I.

Roads.	1. Average Number of Cars Running.	2. Car Mileage.	3. Mean Indicated HP. in Station per Car.	4. Mean Indicated HP. per Car on the Line.	5. Mean Electric HP. in Station per Car.	6. Electric HP. on Car, no Tow.	7. Electric HP. on Car, with Tow.	8. Ratio of Power Motor Car + Tow to Motor Car.	9. Mean Starting Current in Amperes.	10. HP. Hours per Car Mile.	11. Insulation in Megohm Miles.
A	8·24	107	8·1	5·4	6·7	..	..	..	82	0·96	0·15
B	23·4	103	..	..	..	6·2	8·5	1·38	80	0·84	6·9
C	9·0	105	..	..	..	7·7	..	..	47	1·15	3·2
D	14·0	105	6·2	5·2	..	8·6	..	..	75	1·44	0·4
E	8·0	110	8·3	5·3	6·5	8·1	10·3	1·27	80	1·05	0·5
Average	..	106	7·5	5·3	6·6	7·6	9·4	1·33	72	1·09	2·2

TABLE II.

Power on grades.—Average.

A	4 per cent.	17·9 HP.
A	6 "	22·1 "
C	1·43 "	11·9 "
C	3·25 "	15·0 "
C	3·8 "	18·5 "
D	9·5 "	24·7 "

TABLE III.

Cost.—Average of all the four roads in per cent.

Carmen	69·5
Maintenance	11·5
Generating power	19·0
	100·0

Of the power house:—

Labour	11·0
Fuel	6·3
Oil, waste, &c.	1·7
	19·0

E. R. D.

Electrical Subways in New York. By W. MAVER, Jun.

(The Electrical Engineer, March, 1890, p. 84.)

This is a Paper read before the American Institute of Electrical Engineers, and deals very fully with the questions of underground electric-wires, and the practical experience gained with the various systems in use at New York. At the date of the Paper (February, 1890), there were in that city 73 miles of electrical subways, and 1,017 miles of ducts, containing 10,906 miles of telephone-wire; 945 miles of telegraph-wire, and 457 miles of electric-light wires. The total capacity of the present subways is 45,000 miles of telephone- and telegraph-wire, and 2,000 miles of electric-light wires. Access is provided to the subways by manholes, placed at an average distance of 70 yards. The manholes are 5 feet wide, and from 5 to 10 feet deep, built in brick and provided with an iron head-piece and double cover. With the exception of the Brooks and Edison systems, the subways are of the class known as the drawing-in-and-out systems. To draw a new cable in the duct is "rodded"—the technical term for pushing through it a series of wooden rods, jointed together by screws—to establish communication between two successive manholes. A rope is attached to the last rod in the first manhole, and is drawn through to the second manhole, where the sections of the rod are one by one detached. By means of this rope, the cable is finally drawn through.

The Author next describes the construction of the various ducts in detail, and gives an account of the different cables used, and their behaviour after some time of service. As a rule, the insulation is well kept up—the Author citing one instance in which the original resistance of a cable was 1,750 megohms per mile, and after one year's work had only fallen to 1,516 megohms per mile. All the cables used are lead-covered, which is a great protection in handling the cables in the manholes. The greatest difficulty hitherto encountered was the accumulation of gas in the subways; and to minimize the danger of an explosion, blowers have been erected at various points, forcing air through the ducts. As regards the cause of explosions, the Author inclines to the belief that it is not due to the cables, since the latter are, even after an explosion, generally found in perfect condition. Summing up the experience so far obtained in New York, the Author arrives at a series of conclusions, the most important of which is the following: "It is possible to successfully operate all classes of electric conductors underground in cities."

G. K.

The Effect of the Electric Current upon Friction between Metallic Surfaces. By LUCIEN J. BLAKE.

(The Electrical Engineer, New York, 1890, vol. ix., p. 310.)

It has frequently been asserted that the passage of an electric current between a car wheel and the rail on which it runs increases the coefficient of friction between them. As this effect, if true, would be of considerable importance in the application of electrical transmission to tramways, the Author has with various experimental apparatus investigated the subject, with the result that not the slightest difference could be detected in the frictional resistance between two metallic surfaces whether the current was passing or not. The currents employed were not of sufficient intensity to cause heating at the point of contact; and the only contention, therefore, is whether, in electric railways in rapid transit, heating at the contact of wheel and rail could suffice to account for any increase in friction.

F. J.

The Electrical Resistance of the Alloys of Ferro-Manganese and Copper. By EDWARD L. NICHOLS.

(American Journal of Science, 1890, vol. xxxix., p. 471.)

The variations of the temperature coefficient of the high resistance alloys used for resistance standards, though small in any one sample, are still of sufficient importance as regards their effect in observations of precision; and the issue of certain patents for special alloys, for which complete freedom from change of resistance with temperature was claimed, induced Mr. Blood at the Author's suggestion, to undertake an exhaustive investigation into this subject. Alloys containing various percentages of the constituents were carefully made and tested by analysis; and the temperature coefficient for each determined. It was found that some possessed even negative values; which could, however, be changed into positive after subjecting the wire to a process of annealing. The effect, therefore, of the temper of the wire on its coefficient is important, and may introduce variations otherwise unexpected. After repeated applications of annealing from a red heat, the alloys were apparently reduced to a condition of stability. A diagrammatic curve shows the effect on the temperature coefficient introduced by the increase of the proportion of ferro-manganese in the alloy, where the temper of the wire is the same; "between 15 and 20 per cent. the coefficient was very small and crossed the zero line, but it was found that for the entire range from 15 to 30 per cent. the coefficient might be given any value between + 0·00002 and − 0·00002 by varying the temper of the metal."

F. J.

On the Polarization of Electrodes. By L. Poincaré.

(Comptes Rendus de l'Académie des Sciences, vol. cx. 1890, p. 950.)

The maximum polarization of metallic electrodes is a function of the temperature, and by subjecting to experiment salts in a state of fusion, observations can be obtained over a wide range. Thus, with silver electrodes in fused nitrate of soda the polarization falls with extreme regularity from the value of 0·33 volt at 330° Centigrade to 0·1 volt at 440°, and tends towards zero as the temperature is increased and approaches the temperature of decomposition. The same effects are observed with other metals and salts, the fall of polarization being in some cases less regular, and in some instances very rapid in the neighbourhood of the temperature of decomposition. In all cases, however, it may be concluded that when that temperature is reached for any given electrolyte the polarization is zero. This result may possibly be ascribed to the separation of the ions at that temperature, or to some secondary effect arising from a change in the products of the decomposition.

F. J.

The Magnetic Field in the Jefferson Physical Laboratory.

By R. W. Wilson.

(American Journal of Science, 1890, vol. xxxix., p. 456.)

This Paper contains full particulars of a further and more recent determination of the variations in the magnetic intensity of particular rooms in the building,[1] undertaken to discover the effect, if any, of a line of iron water-pipes which had been recently laid near the building, as also the effect of the removal of the iron stove and an iron table, which existed at the time of the earlier observations. These latter were found to have no special effect; but the brick piers were more carefully examined, and were found to have south magnetism uniformly distributed over their upper surfaces, with a magnetic moment of 6,500 C.G.S. units. A soapstone sink in the corner of one room was also a source of considerable disturbance. A microscopic examination of the brick dust and soapstone scrapings proved the existence of crystals of magnetite among other magnetic particles, while a pencil cut from the material of the latter was strongly attracted by a magnet, but without permanent polarization. The disturbing influence of individual bricks was also very marked; and the exclusive use of wood for magnetic observatories is considered as the only safeguard against these difficulties.

F. J.

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[1] Minutes of Proceedings Inst. C.E., vol. c. p. 500.

A Direct-Reading Transmission Dynamometer with Photographic Recording-Apparatus. By Prof. MASCART.

(Comptes Rendus, vol. cx., 1890, p. 605.)

On the shaft of the prime mover are mounted two pulleys, one fast, the other loose. The latter is connected to the former by springs, the deformation of which is indicated by a pointer playing over a scale on one of the pulleys. The scale is empirically determined. When the movement is slow, the position of the pointer can be determined by inspection; but for quick-running shafts, the Author increases the time during which the pointer is visible, by adding to it a lens so placed that the pointer is in its focus. The pointer and corresponding mark on the scale remain then visible during the time that the shaft requires to revolve through an angle equal to the focal angle of the lens. The image of the pointer can be photographed on a slip of sensitized paper set in movement by the shaft. The Author states that the dynamometer described has been fitted to a 10-HP. steam-engine at the Central Meteorological Bureau, but he gives no drawings of the apparatus.

G. K.

On the Ventilation of Buildings.[1] By ALFRED R. WOLFF, M.E.

The Author, after describing in detail the object to be obtained in ventilating a building, commences by refuting the popular notion that the air within the room is composed of two layers, or divisions, the one of vitiated air, the other of pure air, and that the vitiated air, which is presumed to be the heavier, and therefore forms the lower division, can be removed in bulk, leaving the room filled with air of original purity.

The mere mention of the law of the diffusion of gases will suffice to dispel this erroneous view; as it is well known that there is a constant mingling of the vitiated air and the fresh air, depending partly upon their relative temperatures and densities, but mainly on the absolute motion of the air in the room. In other words, a process of dilution of the vitiated air exhaled by man and of the air in the room is constantly going on; and the fresh-air supply must be adequate to keep the air breathed by the inmates at a proper standard of purity.

The Author then further considers the standard of purity of air in any room, and states that the best authorities admit that the proportion of carbonic acid in the air should not exceed in volume 6 to 8 parts in 10,000. It must not, however, be concluded that though carbonic acid itself is a non-supporter of life, it is a poisonous gas in the ordinary acceptance of the term; but it seems strange that in buildings, the carbonic acid given out by the exhalation of man, should not be present in the atmosphere in excess of 6 to 8 parts in 10,000, without making itself felt disagreeably to the senses and being detrimental to health, and the reason of this is, that exhaled with the expired air, besides the carbonic acid, are also organic matters, and aqueous and other vapours, while at the same time there are given off from the pores of the skin organic secretions and moisture.

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[1] The original is in the Library Inst. C.E.

The Author has a very complete formula for determining the quantity of air to be supplied to each inmate in order to secure good ventilation with different proportions of vitiation, together with Tables showing the cubic feet of air containing four parts of carbonic acid in 10,000 necessary to be supplied per person per hour to keep the air in the room at the composition of 6 to 20 parts of carbonic acid in the 10,000, and concludes his remarks on this subject by stating that from 1,500 to 3,000 cubic feet per hour is necessary for theatres and concert halls; for hospitals from 4,000 to 6,000 cubic feet, and for public schools the supply should not be less than 1,800 cubic feet per hour per person.

These amounts of air must be introduced and thoroughly distributed without creating unpleasant draughts, or causing any two points of the room to differ in temperature more than 2° Fahrenheit, and the maximum temperature must not exceed 70° Fahrenheit.

To secure a proper air-supply, the Author further states, is a rare accomplishment, and there are many cases of failure on account of the architects and builders not being sufficiently aware of the need of it; moreover the cost is considerable, even if no money is expended in rendering the ventilating appliances themselves less unsightly.

The question of the cost of removing the vitiated air is next considered, also the area which should be given to the air-ducts under different conditions, and the Author adds a Table, showing the quantity of air in cubic feet discharged per minute through a ventilating duct of which the cross-section area is 1 square foot (the external temperature of the air being 32° Fahrenheit).

In considering the motive power to be used for removing the vitiated air, the Author states his preference for an exhaust-fan or blower, as the same amount of fuel applied to driving this machinery is much more effectively used than in applying it direct to the heating of an upcast-shaft by means of coils or similar methods. He concludes his remarks on this, to the effect that the special methods of air removal, and the proportionating of parts, are matters which necessarily involve so many considerations that they must be left entirely to the engineer who has made a special study of this work.

The Author then considers the several methods of heating used, and is of opinion that the most satisfactory method, as far as fresh air is concerned, is the indirect system by which all the heat entering the rooms represents warm pure air. The registers for admitting the fresh air should be placed above the floor-level, and preferably near the windows, and diagonally opposite the outlet flues, so as to conduce to the thorough distribution of the fresh air to all parts of the room. It is of great importance that the air be not contaminated in transit through the ducts, but that it should be as pure as when it first enters the building.

In conclusion, the Author gives a short notice on the cooling of the air before it enters the building, and he states that the expense being so great, it is rarely resorted to, as the supply of air for 1,000 persons to be lowered 10° Fahrenheit requires at least 1 ton of ice per hour.

W. W. P.

Smokeless Powder. B. N. Experiments made at Sevran-Livry, 27 and 28 March, 1890.

(Revue d'Artillerie, vol. xxxvi., 1890, p. 84.)

These experiments were made with the new quick-firing 15-centimetre (6-inch) gun, system Canet.[1]

With the maximum charge fired, viz., 33 lbs. of B.N. powder, the muzzle-velocity was 2,678 feet per second, and the maximum pressure about 19 tons per square inch.

This gives muzzle energy . . . . 4.609 foot-tons.
„ energy per lb. of powder . . . . 139·7 „
„ „ ton weight of gun . . . 720 „

The following Table gives the results of the trials on 27th and 28th March, 1890 :—

POWDER. B. N.

No. of Round.	Charge. Lbs.	Velocity at 210 Feet from Gun.	Maximum Pressure. Tons per Square Inch.	Recoil in Feet.
1	17·64	1,624	4·17	1·411
2	22·04	1,913	6·33	1·591
3	26·45	2,244	11·31	1·660
4	28·65	2,366	13·83	1·647
5	30·85	2,536	16·87	1·719
6	32·00	2.605	16·96	1·729
7	32·00	2,605	17·54	1·726
8	33·00	2,678	18·54	1·723
9	33·00	2,678	18·70	1·739

J. A. L.

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[1] Revue de l'Artillerie, vol. xxxv. pp. 62, 80 and 85.

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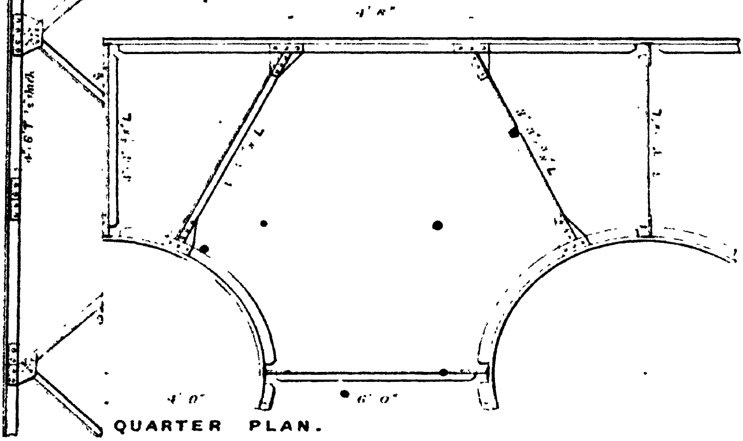

QUARTER PLAN.

Fig. 6b

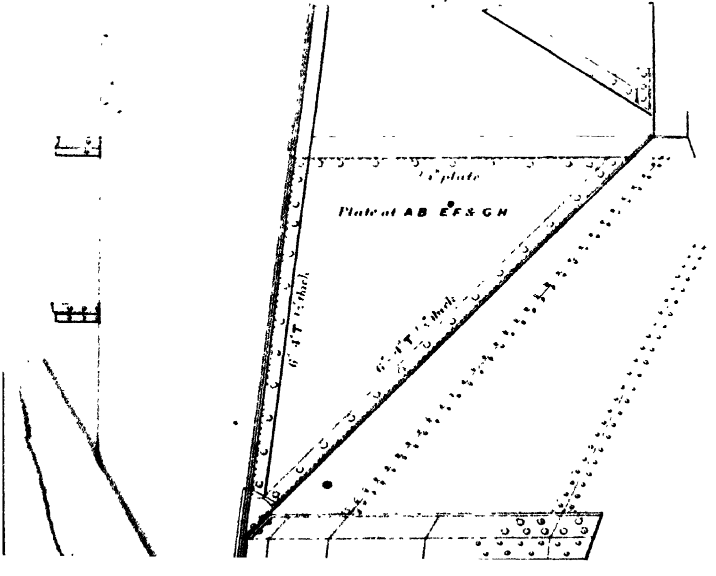

Fig: **RIDGE**

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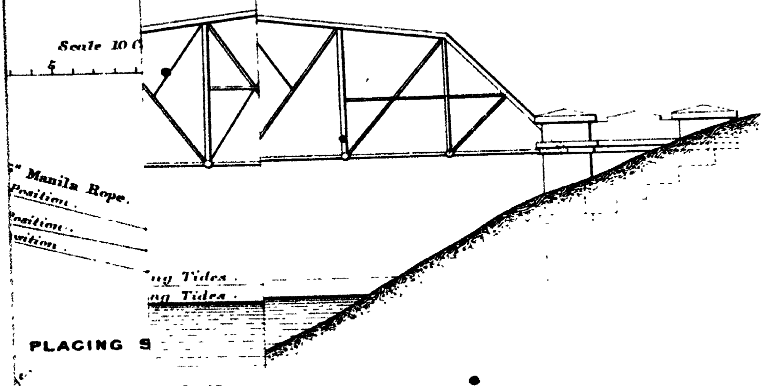

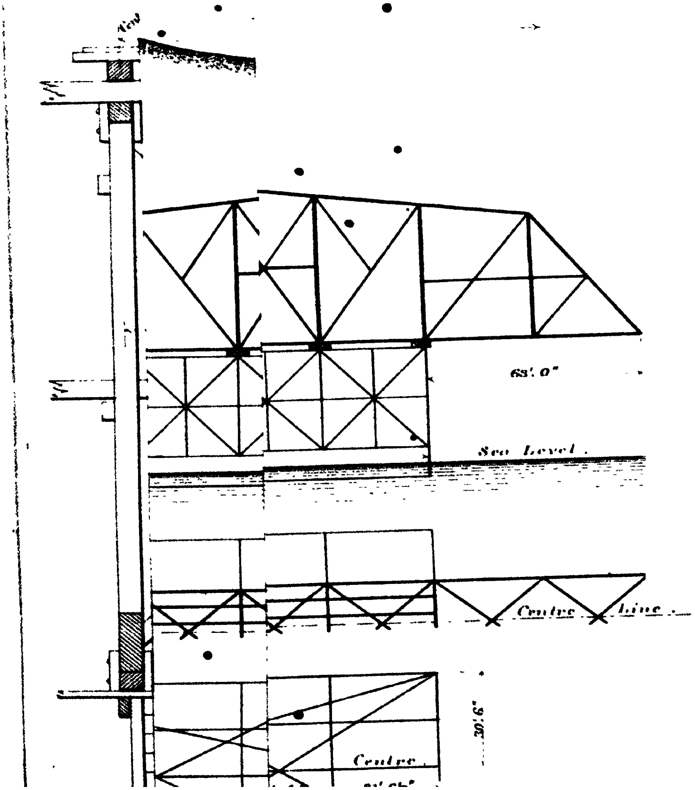

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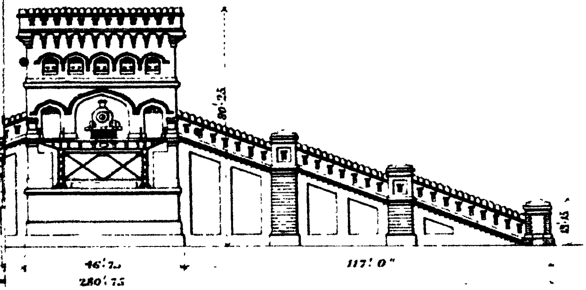

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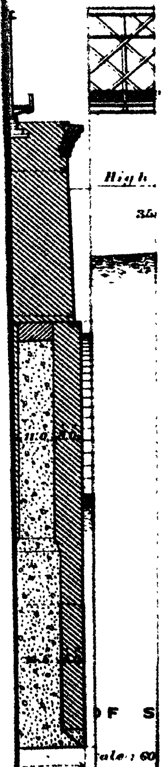

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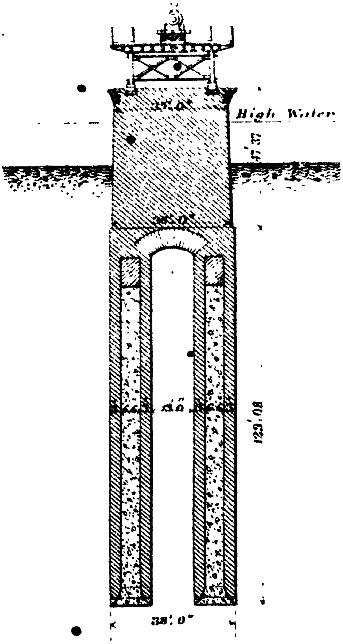
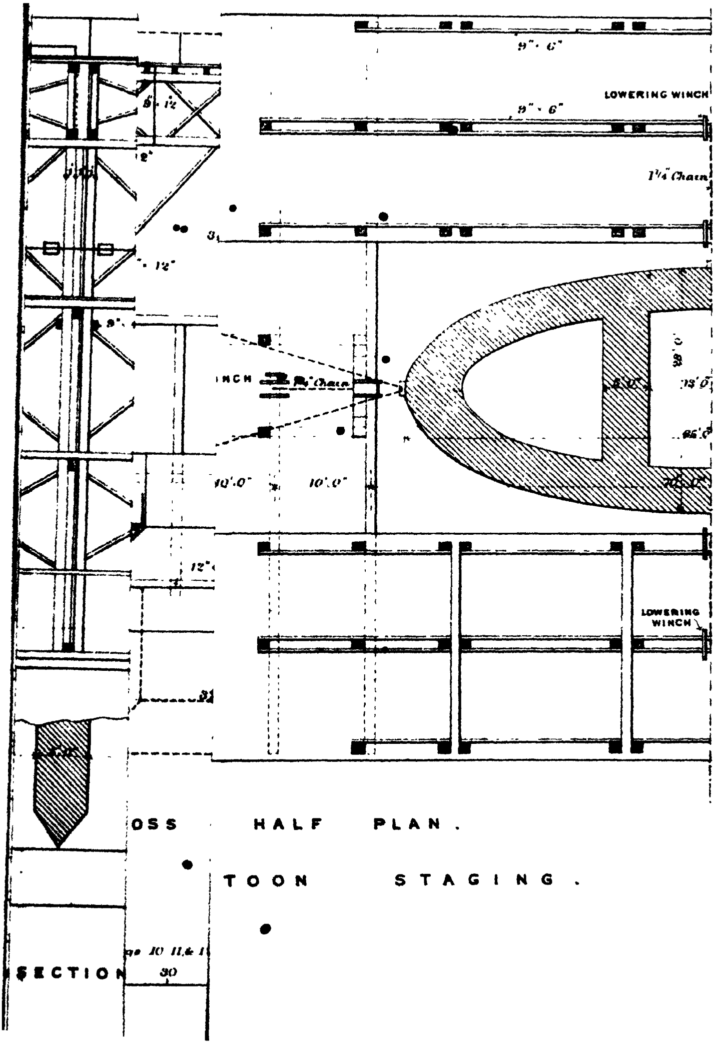


LONGITUDINAL SECTION.

40 KING ST COVENT GARDEN

PLATE. 4.

Fig: 5.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9"·6"
1¼" Chain
8"·12"
12"
9"
INCH
Chain
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10'.0"
6'.0"
13'.0"
66'.0"
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12"
LOWERING WINCH
OSS HALF PLAN.
TOON STAGING.
SECTION
10 11 & 1
30

PLATE. 5.

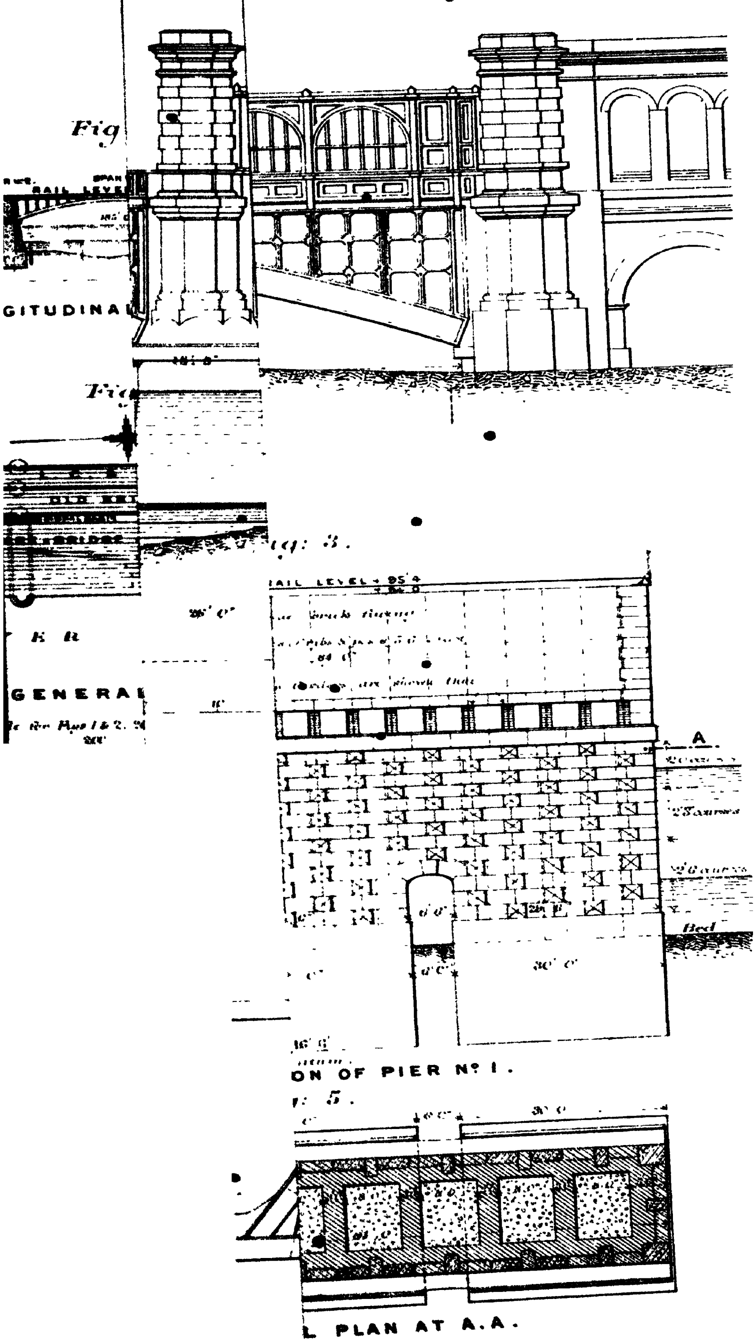
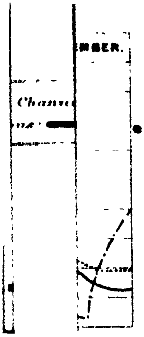


Fig
RAIL LEVEL
SPAN
185' 0
GITUDINAL
181' 6"
Fig
OLD BRI
BRIDGE
E R
GENERAL
20'
26' 0"
Fig: 3.
RAIL LEVEL + 95' 4
+ 84 0
Brick facing
84 0
A
Stream
Bed
28' 6"
30' 0"
16' 0"
ON OF PIER No. 1.
Fig: 5.
30' 0"
L PLAN AT A.A.



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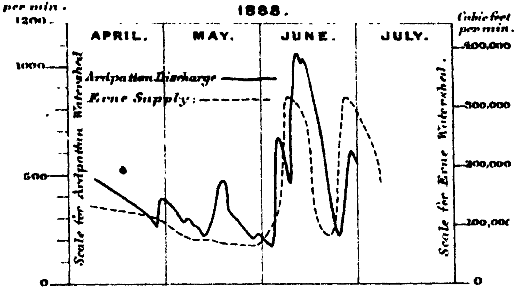
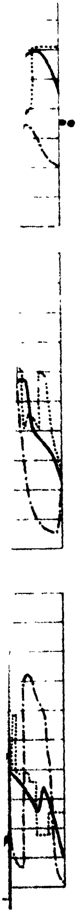


ARDPATTAN DISCHARGE AND ERNE SUPPLY.



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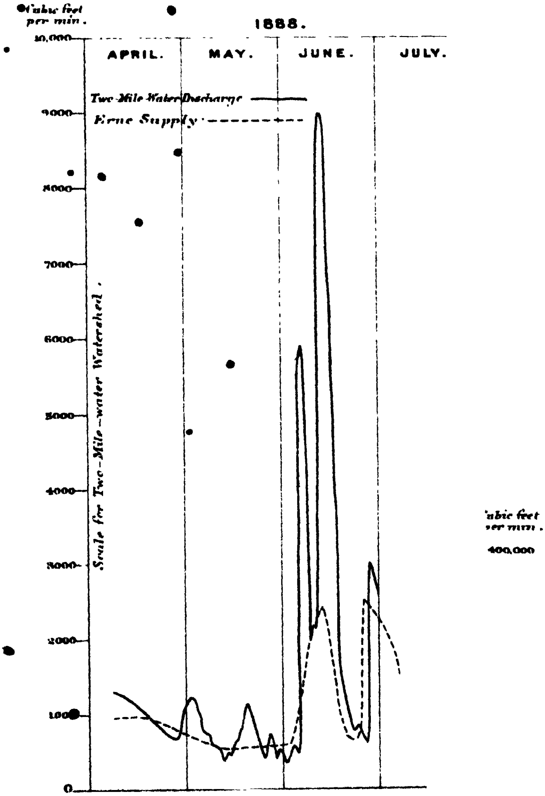


TWO-MILE-WATER DISCHARGE AND ERNE SUPPLY.

Fig: 9. **PLATE. 7.**

AT AA.

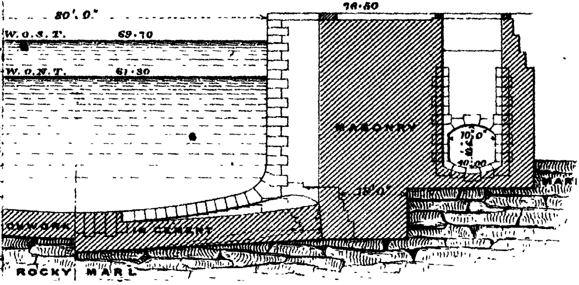
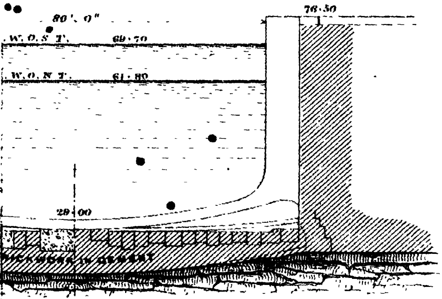


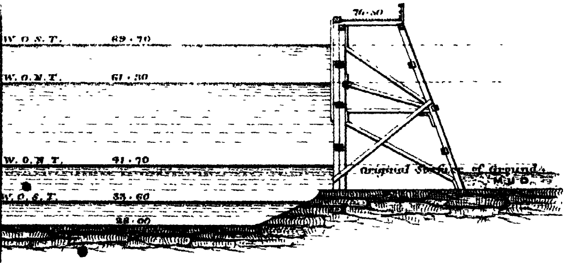
Fig: 10.

AT BB.

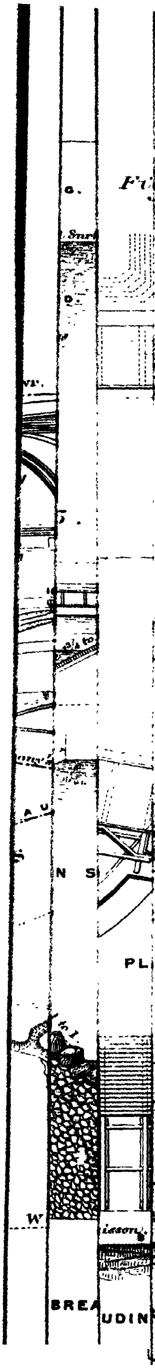


CTIONS OF ENTRANCE.

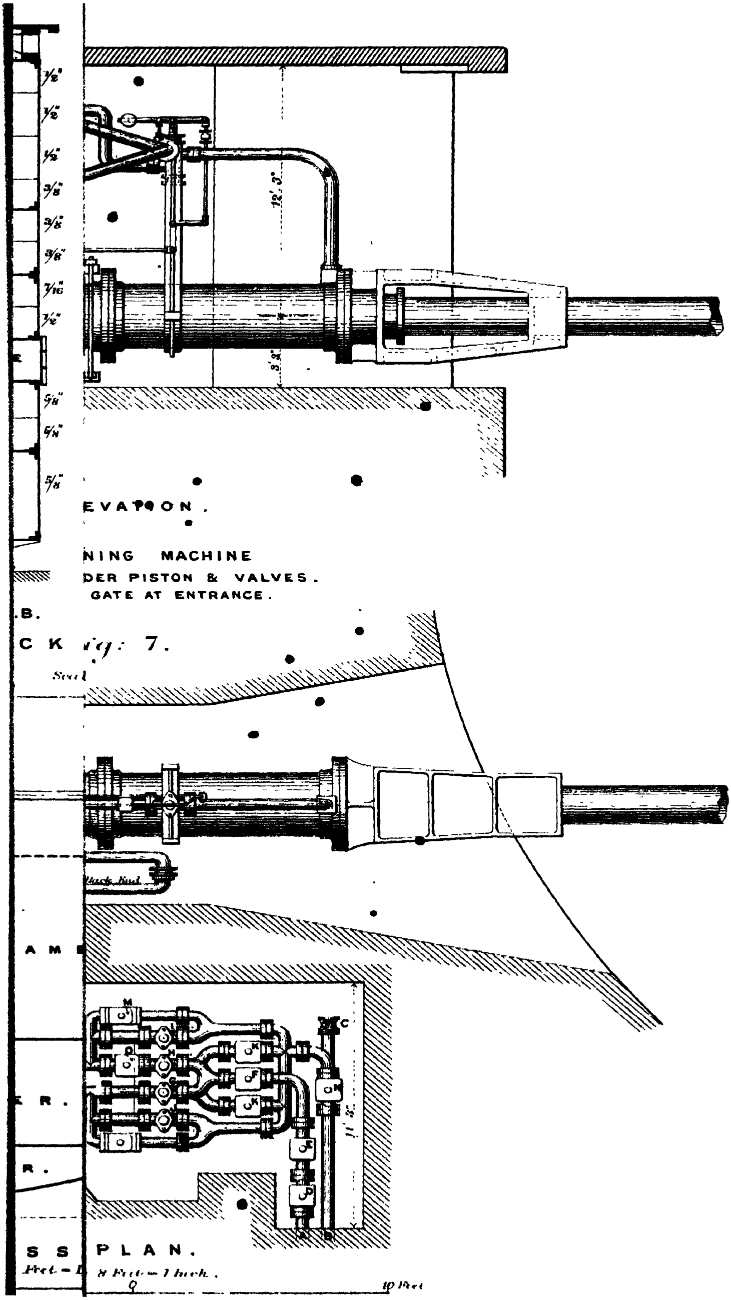
Fig: 11.



ANCE CHANNEL AND JETTIES.



THOs KELL & SON, LITH 40, KING ST COVENT GARDEN


1/2"
1/2"
1/2"
3/8"
3/8"
3/8"
7/16"
1/2"
5/8"
5/8"
5/8"
12' 3"
3' 2"
EVATION.
NING MACHINE
DER PISTON & VALVES.
GATE AT ENTRANCE.
.B.
C K Fig: 7.
Seal
Pack End
A M
R.
R.
M
L
H
G
D
K
F
K
K
C
A
B
11' 3"
S S PLAN.
Feet — I 8 Feet = 1 Inch.
0
10 Feet

PLATE

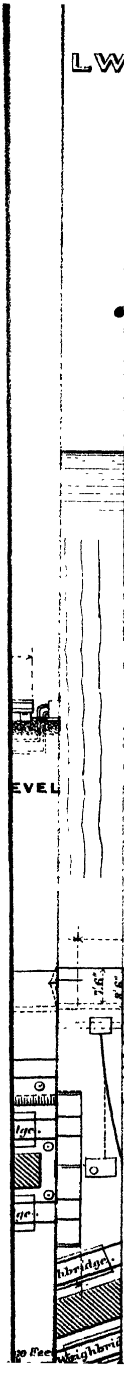
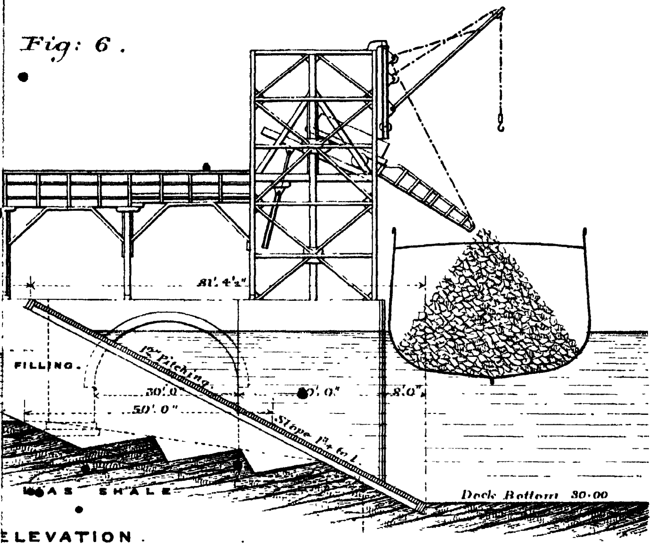


Fig: 6.



ELEVATION.

H LEVEL FIXED TIP.

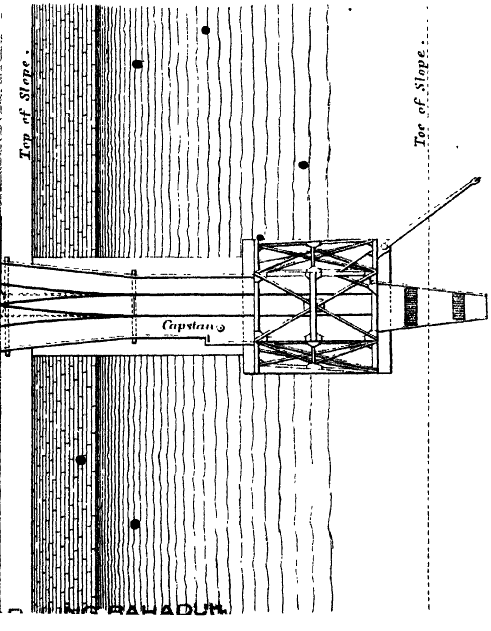
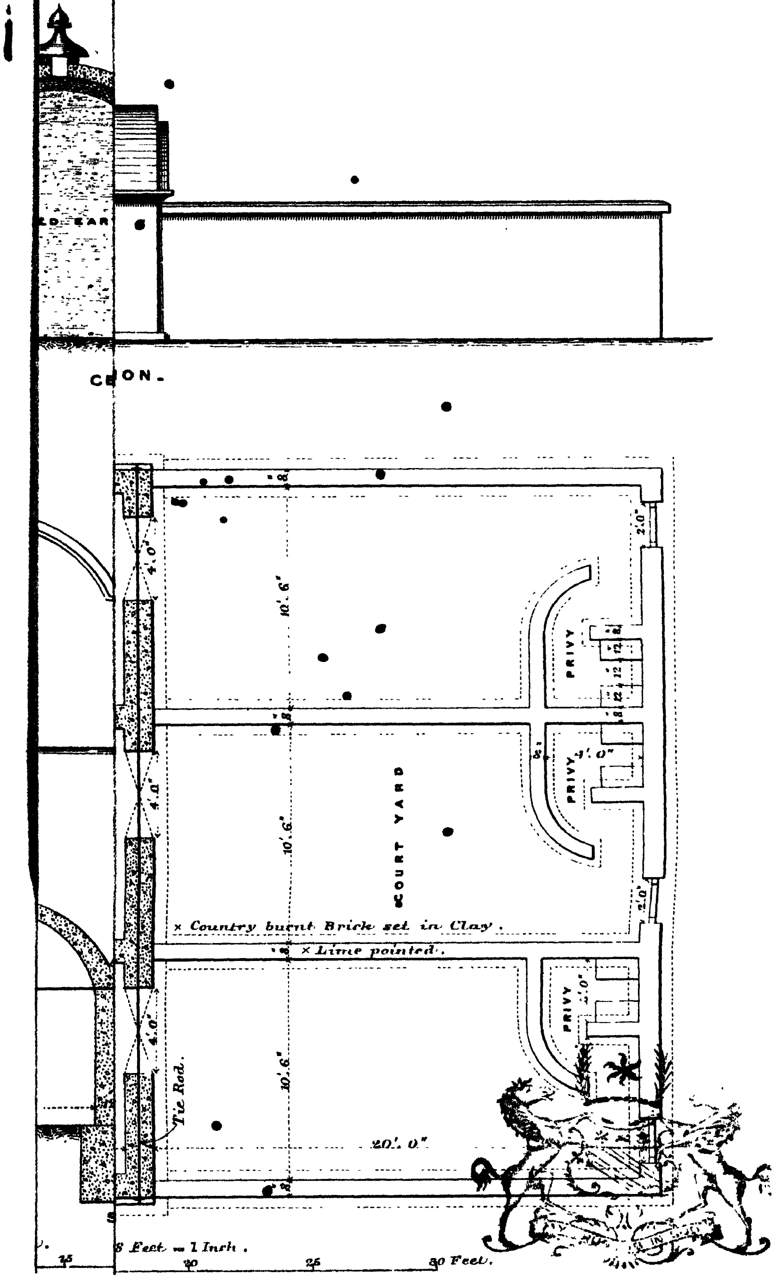


PLATE .10



LD EAR
CE ON.
8"
4'.0"
10'.6"
2'.0"
PRIVY
12
12
12
8
8
4'.0"
PRIVY
COURT YARD
10'.6"
2'.0"
× Country burnt Brick set in Clay.
× Lime pointed.
PRIVY
2'.0"
4'.0"
10'.6"
Tie Rod.
20'.0"
8
8 Feet = 1 Inch.
15
20
25
30 Feet.